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THE DESIGN OF

TYPICAL

STEEL RAILWAY BRIDGES

An Elementary Course for Engineering Students
and Draftsmen

BY

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PREFACE.

This volume may be considered as a sequel to " Bridge and Structural Design," a previous work of the author's; and, like it, has been developed from lectures given by him under the auspices of the Dominion Bridge Company. The structures treated of represent the commonest types of railway bridges, and were chosen because they seemed best suited to illustrate the problems which occur most frequently to the bridge designer. They include the following: a 60-ft. Deck Plate-Girder, a 100-ft. Deck Warren Girder, a 150-ft. Through Pratt Truss, a 200-ft. Through Pratt Truss with curved top chord, a 170-ft. Swing Bridge, and a Railway Viaduct.

At the outset, a general specification has been given governing the loads, unit stresses, and such other details as require the attention of the designer. This is followed by tables for the permissible unit stresses in columns, and for the rivet values used in the designs. After which, instructions have been given for constructing the moment-diagram which is employed in computing the bending moments and shears for wheel-loads. The author has found the moment-diagram to be the most satisfactory method of dealing with wheel-loads, as it affords a rapid and reliable solution, and significant errors are less likely to occur with its use than with that of any form of moment table.

The designs of the different structures are then considered in detail. Commencing with the main dimensions of the structure to be designed, and stating the loads which it has to carry, the stresses in the various members are first determined. The dimensions of these members are then computed, and the details are carefully worked out. In this connection, some knowledge of the strength of materials has of necessity been assumed; but the numerical calculations are given in detail, so that each design is complete in itself. With each design there is an estimate of the weight of the completed structure.

The inclusion of the estimated weights of the structures is an important feature of the book, as estimating forms a large

part of the work of the designer. Where detail drawings are given, the estimates have been figured very closely, and the relative weights of main material and of details are thus accurately obtained. In other cases, a percentage of the weight of the main material in trusses has been added for details. The method of determining this percentage has been clearly demonstrated.

The last chapter is devoted to "The Latticing of Compression Members." This subject is treated from the point of view that the lattice-bars are required to develop the bending stress due to flexure assumed by the particular column formula adopted.

The author trusts that the book may be of service to those in actual practice, as well as to students and draftsmen who are less familiar with the methods used in designing offices.

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CHAPTER I.

INTRODUCTORY.

The structures considered in the following pages have been designed, mainly, in accordance with the Dominion Government specification of 1901, extracts from which are given herewith. A few of the clauses in this specification appear, at first sight, more or less arbitrary, and require some explanation.

Live-Load.—The live-load here specified is one of several classes and is not advocated as a standard, as it would be too light for some railways and too heavy for others; but, for the purpose of illustrating the methods of designing bridges, it is as satisfactory as any.

Impact.—It is generally conceded that a moving load on a structure induces in it greater stresses than a fixed one; and, although it is difficult, if not impossible, to determine these stresses exactly, they may be approximated within reasonable limits.

It can be shown that a load suddenly applied to a structure has twice the effect it would have if applied slowly. Owing to this fact, many specifications allow just one-half the unit stress for live-load stresses which they allow for dead-load stresses. This method of providing for the effect of impact was first employed by Mr. Theodore Cooper, and was a great step in advance of the older method of treating dead-load and live-load stresses identically.

But a train rolling over a bridge is hardly a parallel case to that of a suddenly-applied load. Experiments, made by Professor Turneaure (Transactions, American Society of Civil Engineers, Vol. XLI, page 410), show that the maximum increase of stress in very short spans due to the impact and vibrations caused by a train moving at high speed does not exceed 45% of the stress due to the same load when stationary; and, for spans

of 200 ft., this increase is only about 20%. Then, if the total increase of stress in very short spans (or in members where the dead-load stress = 0), be taken at 100% of the live-load stress, and reduced by some satisfactory formula for longer spans (or for members in which the dead-load stress is proportionately great), it would seem to be ample to cover all contingencies. Cooper's method, which practically adds 100% for impact in all cases, makes the longer spans too heavy in comparison with the shorter ones.

Some specifications really provide for the impact stresses by the use of the Launhardt formula, which gives a different unit stress for each case, depending on the maximum and minimum stresses. This method is not only cumbersome but irrational, being based on the theory of the fatigue of metals which, according to Wöhler's experiments, never occurs in a properly-designed bridge where the stresses are well within the elastic limit. The results, however, are satisfactory so far as the main members of a bridge are concerned; but decidedly unsatisfactory in regard to the details, where the impact stresses are ignored.

The more reasonable, as well as the more convenient, method is to add the impact stresses to those of the dead- and live-loads, and use a uniform unit stress, as for all dead-load. Mr. Waddell, in "De Pontibus," says: "The impact method of proportioning bridges is the only rational and scientifically practical method of designing, even if the amounts of impact assumed be not absolutely correct; for the method carries the effect of impact into every detail and group of rivets, instead of merely affecting the sections of the main members, as do the other methods in common use."

There are several methods in use for computing the impact stresses. First, by fixed percentages of the live-load stresses for the various members of a bridge; which method is inconvenient, as it requires a table; and unsatisfactory because the percentages are usually arbitrary. Second, by a formula taking into account the length of live load which produces the maximum stress in a member, such as that of the American Bridge Co., viz.:

$$I = S\,\frac{300}{L+300},$$

in which I = impact to be added to the live-load stress.
S = calculated maximum live-load stress.
L = length in feet of the loaded distance which produces the maximum stress in each member.

Third, by a formula depending on the ratio of the live-load stress to the total stress in a member, such as that given in the following specifications, viz.:

$$I = \frac{L^2}{L+D},$$

in which I = impact stress.
L = live-load stress.
D = dead-load stress.

This formula seems more rational, as it gives smaller impact stresses in a bridge with a heavy ballast floor than in a bridge of the same span, but having only an ordinary timber floor; whereas, by the American Bridge Co.'s formula, the impact stresses would be the same in both cases. It is also simpler to apply. It was first proposed by Mr. H. S. Prichard, Engineer, New Jersey Steel and Iron Company, in 1895; and later by Mr. E. H. Stone in his paper on the "Determination of the Safe Working Stresses for Railway Bridges" (Transactions, American Society of Civil Engineers, Vol. XLI, page 486). Furthermore, it was strongly advocated by Mr. J. W. Schaub in a paper entitled "Proposed Specifications for Steel Railroad Bridges," and published in the Journal of the Western Society of Engineers, October, 1900; and it was generally upheld in the discussion.

For reversed stresses, the author has slightly modified the formula in order to simplify its application.

Centrifugal Force.—The coefficient 0.02 for centrifugal force is based on an assumed speed of 40 miles per hour on curves up to and including those of 5° of curvature. This coefficient is reduced 0.001 for every degree of curvature above 5°, on the assumption that the speed will be correspondingly decreased. It is obtained from the well-known formula:

$$C = \frac{W v^2}{R g},$$

in which C = centrifugal force.
W = load.
v = velocity of load in feet per second.
R = radius of curve in feet.
g = acceleration of gravity = 32 feet per second.

Thus, for a 1° curve, the radius = 5,730 ft., and the velocity = 40 mi. per hour = 59 ft. per sec. Then

$$C = \frac{W v^2}{R g} = \frac{W \times 59^2}{5{,}730 \times 32} = 0.02\, W.$$

Therefore, the coefficient for centrifugal force $= \frac{C}{W} = 0.02$. For a 10° curve, the radius = 5,730 ÷ 10 = 573 ft. (about), and the coefficient for centrifugal force, according to specification, $= [0.02 - (0.001 \times 5)] \times 10 = 0.15$. The corresponding speed is obtained from the equation

$$0.15 = \frac{v^2}{R g} = \frac{v^2}{573 \times 32};$$

whence

$$v^2 = 0.15 \times 573 \times 32 = 2{,}750, \text{ and}$$
$$v = 52.4 \text{ ft. per sec.} = 35.7 \text{ mi. per hr.}$$

The centrifugal force is considered as part of the live-load in computing the impact.

Longitudinal Force.—The frictional resistance to sliding of wheels on the rails is about 20% of the load. Therefore, this is assumed to be the maximum longitudinal force which a train can exert on a structure. When a train is running freely only the driving-wheels of the engine impart this longitudinal force, which acts in the opposite direction to that in which the train is moving; but, when brakes are put on, every wheel to which the brakes are applied will tend to drag the rails in the same direction in which the train is moving. Thus the latter condition usually gives the greatest longitudinal force.

Combined Direct and Bending Stresses.—When the ties are supported directly by the top chords of a bridge, these top chords are continuous girders over many points of support, and the bending moments due to the wheel-concentrations are difficult to compute accurately. Therefore, in order to simplify

the calculations, the specification requires that the bending moments, both at the panel-points and at the center of panels, shall be assumed equal to three-fourths of the maximum bending moment computed for a simple span of one panel length. This rule, though arbitrary, is entirely on the safe side; and yet it will add very little more to the total section of the chord than would be the case if the bending moments were figured more exactly. In proportioning the chords for bending, the distance from the center of gravity of the section to the extreme outer fibers should be used (whether top or bottom) instead of the distance to the top fibers, as more commonly stated; because, at the panel-points, the bending moments cause compression below the neutral axis, whereas, at the centers of panels, they induce compression above the axis.

Unit Stresses.—For tension and compression, 16,000 lbs. per sq. in. is allowed, which is about one-half the elastic limit of mild steel, thus giving a factor of safety of 2. For compression, the unit stress of 16,000 lbs. per sq. in. is reduced by Rankine's formula, using 18,000 in the denominator for fixed ends; 12,000 for one fixed and one pin end; 9,000 for two pin ends. Both theory and practice go to show that the results obtained by this formula are more nearly correct than those obtained by any straight-line formula; and the only argument in favor of the latter form seems to be that it is easier to apply. But, when a table is used, similar to that following the specification herewith, the Rankine formula is as convenient as any other. In this table (Table I) l = the length of column in feet, and r = the least radius of gyration in inches. Then, to find the permissible unit stress for a given case, it is only necessary to select the value corresponding to the ratio $\frac{l}{r}$. Many modern specifications treat fixed and pin ends identically. This does not seem quite reasonable, as a strut firmly riveted at the ends is undoubtedly much stiffer than one merely connected by pins. For the theory of columns, the reader is referred to "The Theory and Practice of Modern Framed Structures," by Messrs. Johnson, Bryan, and Turneaure.

The bearing values on masonry, and the permissible fiber-stresses for timber may appear rather high at first sight, but it must be remembered that they are to be used in connection with the impact formula.

Plate-Girders.—The specification allows one-eighth of the area of web-plate to be taken as equivalent flange-area, which may be explained as follows:

The gross section modulus of the rectangular web-plate

$$= \frac{b\,h^2}{6} = \frac{b\,h}{6}\,h = \frac{A}{6}\,h,$$

in which b = thickness of web-plate.
h = height of web-plate.
$A = b\,h$ = area of web-plate.

Making allowance for vertical lines of rivet-holes, one inch in diameter and four inches center to center, the net section modulus of the web-plate will be equal to one-eighth of its area multiplied by its height.

The net section modulus of the flanges = $F\,h$,
in which F = net area of one flange.
h = distance center to center of gravity of flanges, and usually assumed to be equal to the height of web-plate when flange-plates are used.

Then, the total net section modulus of girder

$$= \left(\frac{A}{8}h\right) + (F\,h) = \left(\frac{A}{8} + F\right)h.$$

That is to say, the total net section modulus is equal to one-eighth of the area of web-plate plus the net area of one flange, multiplied by the distance center to center of gravity of flanges.

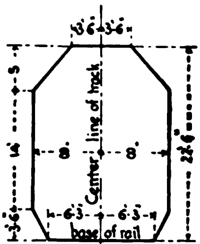
Over the end-bearings and at points of concentrated loading, the stiffeners act as columns. In the first case, the load is received from the web-plate and transmitted to the shoe-plates; in the second case, it is applied at the top of the stiffeners and transmitted to the web-plate. The principal duty of the intermediate stiffeners is to prevent the web-plate from buckling, and to serve as connections for the brace-frames. Mr. Schaub, in his paper already referred to, has shown that, if the load were suspended below the bottom flange, no intermediate stiffeners would be required whatever. His conclusions in this matter were drawn from the following simple experiment, which anyone can easily verify.

A plate-girder model was made of drawing paper, about 15 inches long and 3 inches deep, with two flange-angles top and bottom, and the web-plate projecting below the bottom flange so that loads could be suspended by means of hangers. The girder was supported at the ends, and the top flange stayed to prevent crippling sidewise. It was then found that loads could be suspended from the hangers with perfect safety which could not be placed on top of the girder without buckling the web. Furthermore, the web would stiffen up at once, and, although it was already carrying the suspended loads, additional loads could then be placed on top without causing it to buckle.

Extracts from Dominion Government Specification of 1901 for Steel Railway Bridges.

1. Cross-Ties.—The ties for deck spans, without stringers, shall be at least 8 x 12 ins., or of such depth that a load of 60,000 lbs. on a pair of driving-wheels, distributed over three ties, will not give a greater fiber-stress than that specified hereafter. They shall be 14 ft. long, and spaced 12 ins. center to center.

The ties for through-truss spans shall not be less than 8 x 8 ins. when there are four lines of stringers per track, nor 8 x 10 ins. when there are but two lines. They shall be 14 ft. long, and spaced 12 ins. center to center.



Clearance Diagram

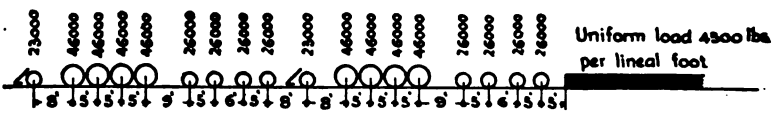
2. Widths.—The width, center to center of through trusses, shall not be less than one-twentieth of span; and, for deck trusses, the width shall not be less than one-fifteenth of span, nor less than 10 ft.

3. Clearances.—The clear open section shall be, on straight track, not less than shown on accompanying diagram. On curved tracks the clear width must be increased to provide the same minimum clearance between cars and trusses.

4. Dead-Load.—The dead-load shall be the weight of the

entire structure, including the floor system; which latter, consisting of the ties, guard-timbers and rails, shall generally be assumed at 600 lbs. per lin. ft. of track.

5. Live-Load.—



or 120,000 lbs. on two axles 7 ft. apart, when this latter loading gives greater stresses.

6. Impact.—For members stressed in one direction only (all tension or all compression), the effect of impact and vibration shall be added to the dead- and live-load stresses, computed by the formula

$$I = \frac{L^2}{L+D}$$

in which I = impact stress.
L = live-load stress.
D = dead-load stress.

For members subject to alternate live-load stresses, the impact shall be computed by the formula

$$I = \frac{range^2}{max.},$$

in which *range* = the numerical sum of the alternate live-load stresses.
max. = the maximum stress due to dead- and live-load which can occur at one time.

This impact shall be considered either as tension or compression, and shall be added to the algebraic sum of the dead-load stress and the live-load tension, or the dead-load stress and the live-load compression. Members shall be proportioned for whichever case requires the greater section.

7. Wind Force.—The lateral bracing in the plane of the unloaded chord shall be proportioned for a wind force of 150 lbs. per lin. ft., uniformly distributed. The lateral bracing in the plane of the loaded chord shall also be proportioned for a uniformly distributed wind force of 150 lbs. per lin. ft., and an additional force of 400 lbs. per lin. ft., to be considered as a moving load and applied 8 ft. above base of rail.

For plate-girders, the lateral bracing shall be proportioned to resist a wind pressure of 30 lbs. per sq. ft. on the exposed surface of one girder and floor, and a moving force of 300 lbs. per lin. ft.

For trestles, the bracing and posts of towers shall be proportioned for a wind force of 300 lbs. per lin. ft., applied 8 ft. above base of rail; also for a force of 30 lbs. per sq. ft. against the exposed surface of one girder and floor, when the structure is loaded, and 50 lbs. per sq. ft. when the structure is unloaded; a force of 225 lbs. per ft. of height on the bents when the structure is loaded, and 375 lbs. per ft. of height when the structure is unloaded.

8. Centrifugal Force.—Centrifugal force for structures on curves shall be taken at 0.02 of the live-load for each degree of curvature up to five degrees. This coefficient (0.02) shall be reduced 0.001 for every degree of curvature above five degrees. The force shall be assumed to be applied at a height of five feet above base of rail.

9. Longitudinal Force.—For trestles, the longitudinal force which is applied to the rails by the traction of engines, or by the sudden application of brakes to a moving train, shall be taken into account. This longitudinal force shall be assumed at 20% of the live-load on structure.

10. Counters.—For counters, 70% only of the dead-load stress shall be assumed to counteract the live-load stress. This rule is intended to provide for a possible future increase in the live-load.

11. Combined Direct and Bending Stresses.—When members are subject to bending moments as well as direct stresses, they shall be designed to resist the combination of the two. In spans with ties resting directly on the chords, the bending moment at center of panels shall be assumed equal but opposite to that at the panel-points, and shall be taken at three-fourths of the moment figured for a simple span of one panel length. Impact shall be added both to the direct and to the bending stresses.

12. Signs for Tension and Compression.—The minus sign shall be used to denote tension, and the plus sign to denote compression.

UNIT STRESSES.

13. Tension and Compression for Mild Steel, 16,000 lbs. per sq. in.; but, for the combination of dead-load, live-load, impact, and wind-force, 20,000 lbs. per sq. in. will be allowed.

In compression, when the length of a member is less than thirty-six times its least radius of gyration, the above unit stress may be used direct; but, when the length exceeds this limit, the unit stress shall be reduced as follows:

For fixed ends, $\dfrac{16{,}000}{1+\dfrac{l^2}{18{,}000\,r^2}}$

For fixed and pin end, $\dfrac{16{,}000}{1+\dfrac{l^2}{12{,}000\,r^2}}$

For pin ends, $\dfrac{16{,}000}{1+\dfrac{l^2}{9{,}000\,r^2}}$

in which l = length of member in ins.
r = least radius of gyration in ins.

No compression member shall have a length exceeding 100 times its least radius of gyration, except wind struts, which may have a length of 120 times their least radius of gyration.

Shearing on bolts, pins, and shop rivets...11,000 lbs. per sq. in.
" on web-plates of girders,........10,000 " " "
Bearing on bolts, pins, and shop rivets....22,000 " " "
For field work, the number of rivets shall be increased 25%.
Bending on outer fibers of pins, 25,000 lbs. per sq. in.
Bearing on rollers per lineal inch, 1,200$\sqrt{\text{diameter}}$.
Bearing of bed-plates on first-class masonry and Portland cement concrete not less than one month old:

Concrete.....................	400 lbs. per sq. in.
Sandstone,....................	300 " " "
Limestone....................	400 " " "
Granite......................	500 " " "

Timber.—Bending on Outer Fibers.—

White oak.....................	1,500 lbs. per sq. in.
Georgia long-leaf yellow pine.......	1,800 " " "
Douglas, Oregon, and yellow fir..	1,800 " " "
Canadian white and red pine......	1,200 " " "

Plate-Girders.

14. Depth.—The depth, back to back of flange-angles, shall be from one-tenth to one-twentieth of span.

15. Width.—The distance center to center of deck plate-girders shall not be less than 8 ft. Through plate-girders shall be spaced far enough apart to give the required clearance.

16. Webs.—The web-plates shall not be less than ⅜-in. thick, and shall have as few splices as possible. One-eighth of the area

of web-plates may be computed as flange material, but in this case the web shall be fully spliced, both for bending and shear.

17. Flanges.—About one-half of the flange-area shall be of angles when practicable.

18. Stiffeners.—When the unsupported distance between the flanges exceeds fifty times the thickness of web, stiffeners shall be used. In girders over four feet in depth, the stiffeners shall not be farther apart than the depth of web-plate. In girders under four feet in depth, they may be four feet apart. Over the end bearings and at points of concentrated loading, the stiffeners shall be designed as columns; and shall have sufficient rivets to transmit the total load to the web-plate. Fillers shall be used with these stiffeners. Intermediate stiffeners, which may be crimped, shall be as follows:

For webs	3	ft. deep and under, two	$3 \times 2\frac{1}{2} \times \frac{5}{16}$-in.	angles
" "	4	" " " " "	$3 \times 3 \times \frac{5}{16}$-in.	"
" "	5	" " " " "	$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{5}{16}$-in.	"
" "	6	" " " " "	$4 \times 3\frac{1}{2} \times \frac{3}{8}$-in.	"
" "	7	" " " " "	$5 \times 3\frac{1}{2} \times \frac{3}{8}$-in.	"

Floorbeams and Stringers.

19. Spacing of Stringers.—Stringers shall be spaced in general 8 ft. apart center to center. When there are four lines of stringers, they shall be spaced 3 ft. apart. The inner lines shall be figured for two-thirds of the load, and the outer lines for one-third.

20. End Angles.—The end-connection angles of floorbeams and stringers shall not be less than one-half inch thick, and they shall have fillers under them with at least one row of rivets outside those in the angles.

21. Lateral Bracing.—The stringers shall be braced laterally when their length exceeds fifteen times the width of flanges.

Details.

22. Minimum Sections.—No material thinner than $\frac{3}{8}$-in. shall be used, whether in plates or shapes, except lattice-bars, stiffeners, and wind-bracing. The latter shall not be less than $3 \times 2\frac{1}{2} \times \frac{5}{16}$-in. angles. No rod shall have a less area than one square inch.

Angles, cover-plates, and web-plates shall be as long as practicable to avoid splicing. The unsupported width of any plate, subject to compression, shall never exceed thirty times its thickness, except for cover-plates of top chords and end posts, when it may be forty times its thickness.

23. Tie-Plates and Lattice-Bars.—The open sides of all compression members shall be stayed by tie-plates at the ends, and lattice-bars between. The thickness of tie-plates shall not be

TABLE I.—PERMISSIBLE COMPRESSION PER SQ. IN. FOR COLUMNS.
Dominion Government Specification, 1901.

Square Ends $\dfrac{16000}{1+\dfrac{(12l)^2}{18000r^2}}$ Square and Pin End $\dfrac{16000}{1+\dfrac{(12l)^2}{12000r^2}}$ Pin Ends $\dfrac{16000}{1+\dfrac{(12l)^2}{9000r^2}}$

$\frac{l}{r}$	Square Ends	Square and Pin End	Pin Ends	$\frac{l}{r}$	Square Ends	Square and Pin End	Pin Ends
3.0	14,920	14,440	13,990	7.8	10,760	9,250	8,110
3.2	14,790	14,250	13,760	8.0	10,580	9,060	7,910
3.4	14,650	14,060	13,500	8.2	10,400	8,860	7,710
3.6	14,500	13,860	13,280	8.4	10,230	8,660	7,820
3.8	14,380	13,640	13,000	8.6	10,060	8,480	7,330
4.0	14,190	13,420	12,740	8.8	9,880	8,300	7,150
4.2	14,020	13,210	12,480	9.0	9,710	8,110	6,970
4.4	13,860	12,990	12,220	9.2	9,540	7,940	6,800
4.6	13,690	12,760	11,980	9.4	9,370	7,770	6,630
4.8	13,510	12,530	11,700	9.6	9,210	7,600	6,470
5.0	13,330	12,310	11,430	9.8	9,050	7,430	6,310
5.2	13,160	12,080	11,170	10.0	8,890	7,270	6,150
5.4	12,980	11,850	10,910	10.2	8,730	7,120	6,010
5.6	12,800	11,620	10,660	10.4	8,570	6,960	5,860
5.8	12,610	11,400	10,400	10.6	8,430	6,820	5,720
6.0	12,430	11,170	10,150	10.8	8,280	6,670	5,580
6.2	12,240	10,950	9,910	11.0	8,130	6,530	5,450
6.4	12,050	10,730	9,670	11.2	7,990	6,390	5,320
6.6	11,870	10,510	9,430	11.4	7,860	6,250	5,200
6.8	11,680	10,290	9,200	11.6	7,710	6,120	5,080
7.0	11,500	10,080	8,970	11.8	7,570	5,990	4,960
7.2	11,310	9,860	8,760	12.0	7,440	5,870	4,840
7.4	11,130	9,660	8,530	12.2	7,310	5,740	4,730
7.6	10,940	9,460	8,320	12.4	7,180	5,620	4,620

TABLE II.—SHEARING AND BEARING VALUE OF RIVETS, IN POUNDS.
Dominion Government Specification, 1901.

Diameter of Rivet		Area in Square Inches	Single Shear at 11000 lbs.	Bearing Value for Different Thicknesses of Plate at 22000 lbs.												
Fraction	Decimal			$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
$\frac{3}{8}$	.375	.1104	1210	2060	2580	3090										
$\frac{1}{2}$	.500	.1963	2160	2750	3440	4130	4820	5500								
$\frac{5}{8}$	.625	.3068	3370	3440	4300	5160	6020	6880	7740	8600						
$\frac{3}{4}$	.750	.4418	4860	4130	5160	6190	7220	8250	9280	10320	11340	12380				
$\frac{7}{8}$	.875	.6013	6610	4810	6020	7220	8430	9630	10840	12040	13240	14440	15640	16840	18050	
1	1.000	.7854	8640	5500	6880	8250	9630	11000	12380	13750	15130	16500	17880	19250	20630	22000

less than one-fiftieth of the unsupported width, unless the edges are supported by angles. The length of tie-plates on each side of intermediate panel-points shall not be less than twelve inches; and, at other points, the length shall not be less than the width of member.

Single lattice-bars shall have a thickness of not less than one-fiftieth of their unsupported length. The distance between connections of lattice-bars shall be such that the individual members composing column, considered as columns of a length equal to the distance between these connections, shall be stronger than the column as a whole. Ordinarily, single latticing shall make an angle of 60° with the axis of member, and double latticing an angle of 45°.

The minimum widths of lattice-bars shall be as follows:

For	members	8 ins.	deep	and	under,	$1\frac{3}{4}$-in.	bars	shall	be used.
"	"	14 ins.	"	"	"	$2\frac{1}{4}$-in.	"	"	"
"	"	20 ins.	"	"	"	4 -in.	"	"	"

The 4-in. bars shall have two rivets at each end.

24. Rivet Spacing.—The pitch of rivets shall not be less than three diameters of rivet, nor greater than 6 ins. At ends of compression members the pitch shall not be greater than four diameters of rivet for a distance equal to twice the depth of member. In flanges of girders and chords supporting the ties, the pitch shall not exceed 4 ins. The center of a rivet shall not be nearer to the edge of metal than $1\frac{1}{4}$ ins., except in bars under $2\frac{1}{2}$ ins. wide. When practicable, the distance shall be at least two diameters of rivet, and it shall not exceed eight times the thickness of plate. The distance between centers of rivets in compression members shall not exceed sixteen times the thickness of the thinnest outside plate or shape, in line of stress; nor forty times the thickness, at right angles to line of stress.

25. End Bearings.—All spans of 75 ft. and over shall have rollers at one end, not less than 5 ins. in diameter. Spans under 75 ft. shall have planed sliding surfaces at one end to allow for expansion. Bed-plates shall be of sufficient thickness to distribute the load evenly over masonry, and shall not be less than $\frac{7}{8}$-in. thick.

MOMENT-DIAGRAM.

For finding reactions, panel-concentrations, shears, and moments due to wheel-loads, a graphical method will be used, which is here recommended on account of its simplicity, accuracy, and universal adaptability. The stresses obtained in this manner may not be quite as exact as if they were computed analytically, yet they are close enough for all practical pur-

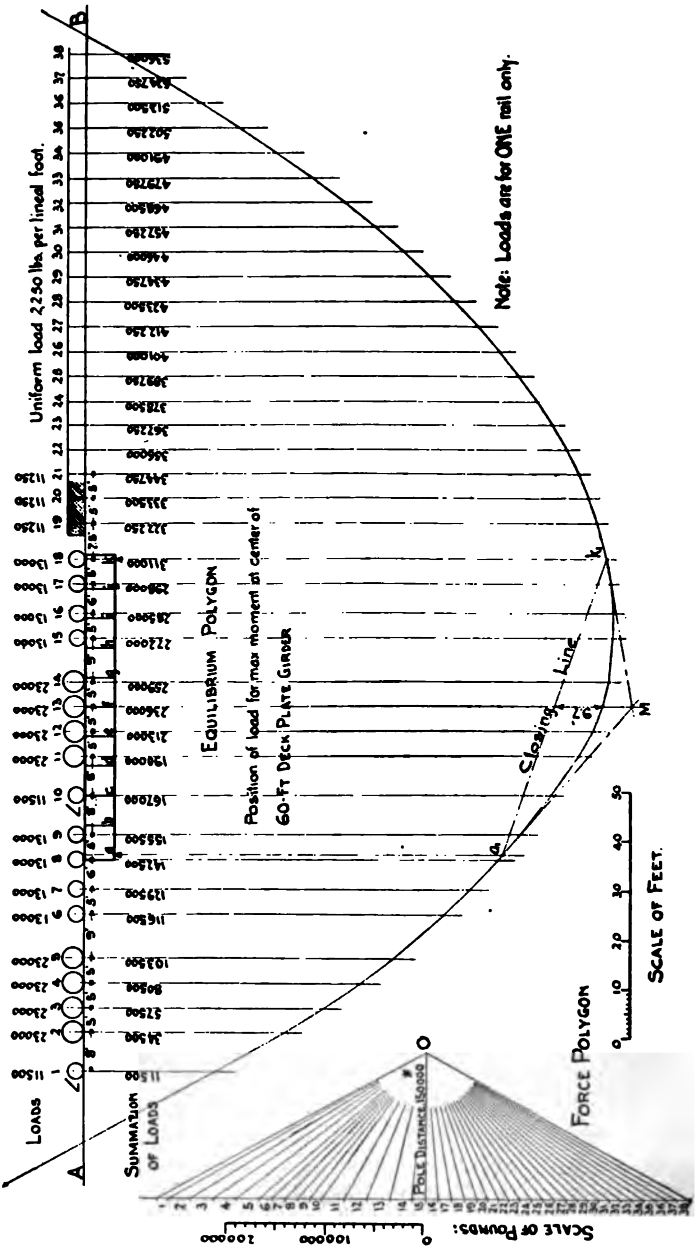


Fig. 1.—Moment-Diagram.

poses, and the chances of significant errors are reduced to a minimum. A moment-diagram for a given set of wheel-loads can be constructed in much less time than a moment-table, its application is simpler, and the liability of mistakes with it is proportionately less. A moment-diagram can also be prepared more easily than a table of so-called equivalent uniform loads, its application is quite as simple, and the results obtained by it are more exact.

For analytical methods of dealing with wheel-loads, the student is referred to Johnson's "Theory and Practice of Modern Framed Structures," before noted; and, for the theory of the moment-diagram, he is referred to Greene's "Graphics."

Directions for constructing the moment-diagram are given herewith, as follows:

On the line *A B* (Fig. 1), the locomotive-wheels from 1 to 18 are spaced as shown; and these are followed by the uniform load, assumed to be concentrated at 5-ft. intervals. The loads, which are for one rail only, are given above the line; and their summation, below. A scale of 10 ft, to the inch will be found to give satisfactory results: a smaller scale is hardly accurate enough, and a larger one makes the diagram unwieldy.

The force-polygon is drawn next: On a vertical line, at the left of the figure, the loads are laid off in regular order to a convenient scale of pounds, and numbered to correspond with those on the line *A B*. The point *O*, which is called the pole, may be taken anywhere to the right of the vertical, but preferably about opposite its middle point, and distant from it horizontally some even number of pounds, measured by the same scale as that used for the loads. The best results will be obtained when the pole-distance is equal to about one-quarter the length of load-line. In the figure, it is taken at 150,000 lbs. From the pole, lines are drawn radially to the points of division on the load-line, representing the loads 1, 2, 3, etc.

The equilibrium-polygon will now be constructed: Beginning at any point on the vertical line through wheel 1, a line of indefinite length is drawn to the left, parallel with the radial line which passes through the upper extremity of load 1 in the force-polygon. From the same point on the vertical through wheel 1, a line is drawn to the right, parallel with the radial line which passes through the lower extremity of load 1 in the force-

polygon, and intersecting the vertical through wheel 2. From the last intersection, a line is drawn to the right, parallel with the radial line which passes through the lower extremity of load 2 in the force-polygon, and intersecting the vertical through wheel 3. And so on to the end. It should be observed that the lines drawn to the left and right of a vertical through a wheel-concentration are parallel with the radial lines which pass through the upper and lower extremities of the corresponding load in the force-polygon.

In order to insure accurate results, the moment-diagram should be constructed with great care on white drawing paper, using fine ink-lines. Its use will be demonstrated in the following chapters.

CHAPTER II.

THE DESIGN OF A 60-FT. DECK PLATE-GIRDER.

The deck plate-girder is one of the commonest, as well as one of the most useful, types of railway bridges. It is suitable for spans of from about 20 ft. to 90 or 100 ft. Plate-girders considerably longer than 100 ft. have been built, but they are seldom economical. The chief advantages of the plate-girder are its rigidity and lasting qualities. Its rivets seldom work loose, and it is easy to paint. As a permanent structure, it ranks next to the masonry arch. Although comparatively simple to construct, the stresses in a plate-girder are somewhat complex; and there are many points in its design which require careful attention, the greater number of which will be dealt with in the following example.

One of the first things to consider in designing a bridge is the dead-load. The weight of the ties, guard-timbers, and rails can be easily calculated, or their weight is frequently specified, as in the present case. The weight of the metal in the structure, however, must be assumed, as it cannot be determined exactly until the design is finished; and, on completion, if it should be found that the actual weight differs materially from that assumed, it would be necessary to revise the computations. But this is seldom necessary in practice, as the weight of metal can usually be estimated closely enough from the known weights of similar structures. For the loads and unit stresses used in this design the following formula will be found to give fairly close results:

$$w = 10\,l + 100,$$

in which w = weight of metal per lineal foot.
l = length of span in feet.

For a heavier live-load, or for lower unit stresses, the factor (10) would have to be increased. Using this formula, the weight of metal in the span under consideration will be

$$(10 \times 60) + 100 = 700 \text{ lbs. per lin. ft.}$$

The principal data for the design will now be as follows:

Length, 60 ft. c. to c. of end-bearings.
Depth, 6 ft. back to back of flange-angles.
Width. 8 feet c. to c. of girders.
Dead-load (floor), 600 lbs. per lin. ft.
(steel), 700 " " " "
(total), 1,300 " " " "
Live-load, as per specification.

The dead-load shears and moments will be computed analytically by the well-known formulas given below:

Reaction or end shear $= \frac{w\,l}{2}$,

Shear at intermediate section $= \frac{w\,l}{2} - w\,x$,

Moment at center $= \frac{w\,l^2}{8}$,

Moment at intermediate point $= \frac{w\,l\,x}{2} - \frac{w\,x^2}{2}$,

in which w = uniform load per lineal foot.
l = length of span, c. to c. of bearings.
x = distance from end to point where shear or moment is required.

For the present, only the end shear and the center moment will be required. The dead-load for one girder = **1,300÷2 = 650** lbs. per lin. ft.

Dead-load end shear $= \frac{650 \times 60}{2} = 19{,}500$ lbs.

Dead-load moment at center $= \frac{650 \times 60^2}{8} = 292{,}000$ ft.-lbs.

For the live-load shears and moments, the moment-diagram (Fig. 1) will be used. A diagram of the girder should be constructed on tracing cloth, to the same scale as the equilibrium-polygon, and divided into any number of equal panels, with the panel-points lettered for reference. In the example, the girder is divided into ten 6-ft. panels. The tracing cloth should

be large enough so that the closing lines and other marks can be made on it, and thus avoid marking on the moment-diagram.

Maximum Reaction, or End Shear.—Since every load on the span contributes to the end reaction, it follows that, for the maximum reaction, the live-load should cover the span, with the heavier loads as near as possible to the end considered. Therefore, the first driving-wheel (wheel 2) should be placed

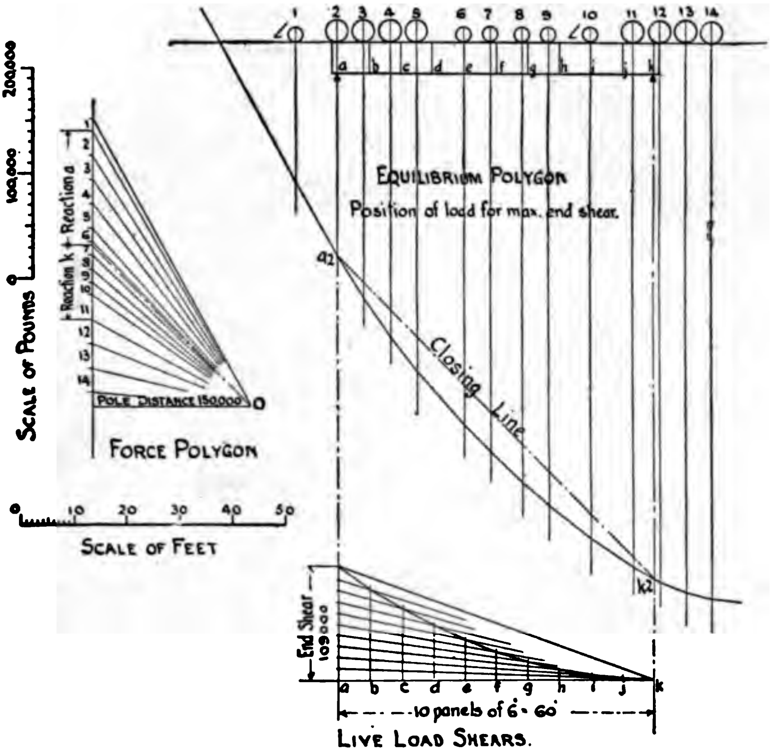


FIG 2—60-ft. Deck Plate-Girder.

over the end a. The diagram of the girder should be brought to this position, as shown in Fig. 2, and verticals dropped from the ends a and k to the equilibrium-polygon, intersecting it in the points a_2 and k_2. Then a line drawn through these last-named points is the closing line for that portion of the equilibrium-polygon which belongs to the loads on the span; and a line drawn from the pole O in force-polygon, parallel with the

closing line, and intersecting the vertical load-line, determines the reactions at both ends. The upper part, which measures 109,000 lbs., is the reaction at a; the lower part, the reaction at k, as shown.

Having found the maximum end shear, the maximum shear at any point can readily be determined, with sufficient accuracy, by constructing a parabola with apex at k, and the maximum ordinate at a equal to the end shear, as shown in Fig. 2. The maximum live-load shear at any point will then be the ordinate to the parabola at that point.

Maximum Moment.—For the maximum moment, the greatest possible load should be placed on the span, with the heavier wheels near the center. Then, if the center of gravity of the total load on span coincide with a wheel-concentration, the maximum moment will occur under this wheel, when placed at the center of span. If the center of gravity of the load fall between two wheels, the maximum moment will be under one of these (usually the wheel nearest the center of gravity), when the wheel considered and the center of gravity are equidistant from the center line of span. It may be necessary to try both of the wheels adjacent to the center of gravity to determine the maximum moment.

With wheel 13 at the center, as shown in Fig. 1, it is evident that the greatest possible load is on the span. The center of gravity of loads 9 to 17, inclusive, is found by extending the outer sides of that portion of the equilibrium-polygon which belongs to these loads to an intersection in the point M, which is directly below wheel 13; and, consequently, this load is at the center of gravity, and this position will give the maximum moment. Wheel 18, which is at the point of support k, is not included in the above system of loads because it does not affect the bending moment.

From the points of support a and k, verticals are dropped, intersecting the equilibrium-polygon in a_1 and k_1, and the closing line is drawn through these last-named points. Then, the moment at any point of the span for this position of the load is proportional to the ordinate, measured vertically, between the closing line and the equilibrium-polygon; and is equal to the ordinate multiplied by the pole-distance in force-polygon. In the example, the ordinate at the center of span measures

9.7 ft.; then, 9.7 ft. × 150,000 lbs. = 1,455,000 ft.-lbs., which is the maximum moment.

The shears and moments will now be summarized, with the impacts added, according to specification.

End Shear.—Dead-load .... = 19,500
Live-load .... = 109,000

$$\text{Impact} = \frac{109{,}000^2}{109{,}000 + 19{,}500} = 93{,}400$$

221,900 lbs.

Unless otherwise specified, the permissible unit shear for web-plates is usually assumed to be on the gross area. In the present case it is 10,000 lbs. per sq. in.

Area of web-plate required = 221,900 ÷ 10,000 = 22.19 sq. ins. A plate 72x$\frac{3}{8}$-in. will be used throughout. Its area = 27 sq. ins.

Center Moment.—Dead-load = 292,000
Live-load = 1,455,000

$$\text{Impact} = \frac{1{,}455{,}000^2}{1{,}455{,}000 + 292{,}000} = 1{,}190{,}000$$

2,937,000 ft.-lbs.

The effective depth of a plate-girder is the distance c. to c. of gravity of the flanges; therefore it cannot be known exactly before the section of the flanges has been decided on. When the flanges are composed of two angles with cover-plates, it will usually be found that the effective depth is approximately equal to the depth back to back of angles; so this depth may be used in preliminary calculations to determine the flange section. The center of gravity of the flanges can then be readily computed, and the correct effective depth thus obtained. If this effective depth is found to be appreciably less than the distance back to back of angles, the calculations should be corrected accordingly; but, if greater than the distance back to back of angles, then this latter distance should be assumed as the effective depth. This is in conformity with general practice.

Returning to the example, the effective depth will be assumed as 6 ft. Then the flange-stress at center of girder = 2,937,000

ft.-lbs. ÷ 6 ft. = 489,500 lbs.; and the net section required for the bottom flange = 489,500 ÷ 16,000 = 30.6 sq. ins.

The make-up of the bottom flange will now be considered. According to specification, and as more fully explained in the introductory remarks thereto, one-eighth of the area of the web-plate may be considered as equivalent flange-area. In determining the net section of flange material, the diameter of the rivet holes will be assumed ⅛-in. larger than that of the rivet before driving; for the actual diameter of the hole is $\frac{1}{16}$-in. greater than that of the rivet, and the extra $\frac{1}{16}$-in. is to allow for injury to the metal around hole, due to punching. In general, when there are rivet holes in both legs of an angle, or if there are two lines of rivets in one leg, staggered two inches or less, two holes should be allowed for. If there is a single line of rivets in one leg of angle, or if there are two lines, staggered more than two inches, then one hole only need be allowed for. In cover-plates with two lines of rivets, allowance should be made for two holes. In the example, ⅞-in. rivets will be used, and thus the diameter of holes will be assumed as 1 in

The flange-section at center will be as follows:

⅛ of 72 x ⅜-in. web-plate	=	3.37
2 angles, 6 x 6 x ⅝-in. = 14.22 sq. ins., gross; less 4 holes 1 in. in diam.,	=	11.72
2 plates, 15 x ⅝-in. = 18.26 sq. ins., gross; less 4 holes 1 in. in diam.	=	16.26
		31.35 sq. ins. net area.

It will be found that the center of gravity of this section practically coincides with the back of the angles, thus the assumed effective depth is correct.

The usual method of determining the lengths of the cover-plates will now be demonstrated: On a horizontal line projected below the diagram of the girder (Fig. 3), a parabola is constructed with vertex at center of span, and the maximum ordinate at this point equal to the required flange-area, viz., 30.6 sq. ins. Then, the flange-area required at any point will be represented by the ordinate to the parabola at that point. The flange material is then superimposed on the parabola, the net areas of the component parts being represented to the same

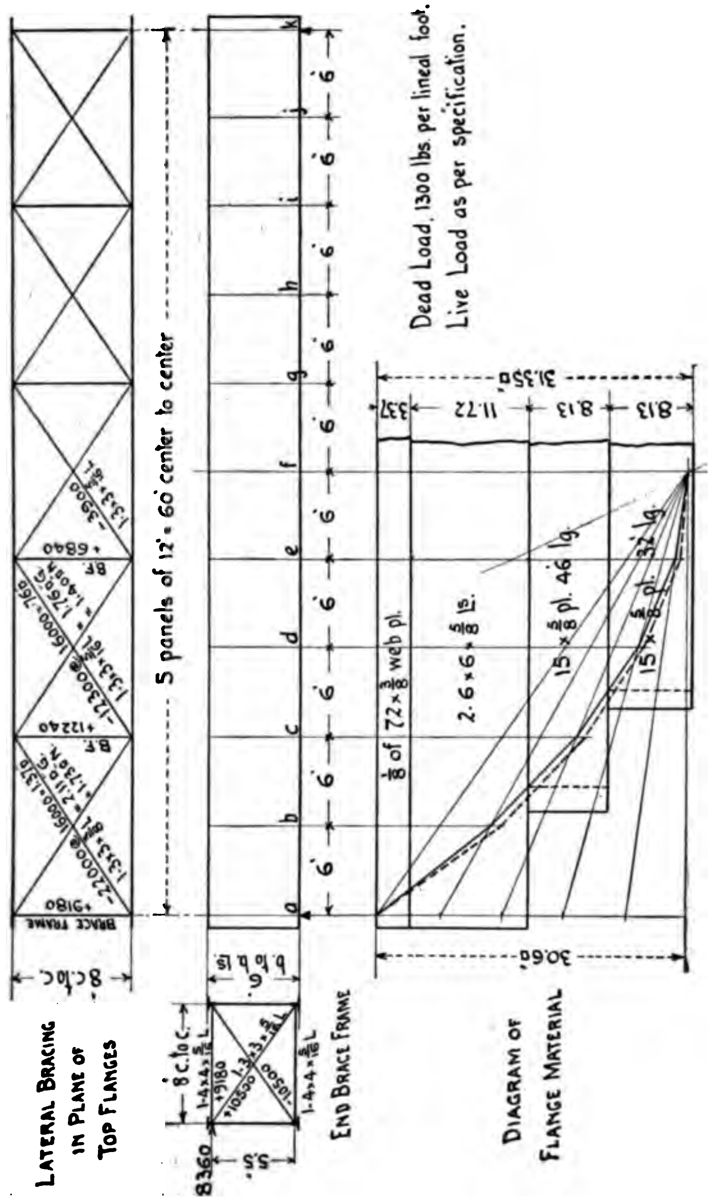


Fig. 3.—60-ft. Deck Plate-Girder.

vertical scale used for the required areas. The first rectangle, representing the equivalent flange-area of the web-plate, and the second rectangle, representing the net area of two flange-angles, extend the full length of girder. The third rectangle, representing the net area of the first cover-plate, is required theoretically to the point where the parabola cuts the lower boundary of the second rectangle; and the fourth rectangle, representing the net area of the second cover-plate, is required theoretically to the point where the parabola cuts the lower boundary of the third rectangle. Both of the cover-plates, however, are extended about one foot beyond the theoretical points to which they are required, in order to compensate for the somewhat greater bending moments towards the ends of the girder due to the wheel-loads in different positions, and for the less effective depth of girder, due to stopping off the cover-plates. Of course, the theoretical lengths of cover-plates can be computed analytically by a formula based on the equation of the parabola; but the graphical method here shown is recommended, because errors are less likely to occur in this way.

A more exact method of determining the lengths of cover-plates (which, however, requires a good deal of labor) is to compute the flange-section required at regular intervals, taking into account the maximum bending moments and the variations in the effective depth; then, using the required areas as ordinates, to plot a curve of areas, which replaces the parabola. The flange material is then laid on as before.

To illustrate: It will be found that the area required at b = 12.1 sq. ins., at c = 20.5 sq. ins., at d = 26.3 sq. ins., at e = 30.2 sq. ins. Using these values as ordinates, the curve shown in dotted lines, below the parabola, is obtained. It will be observed that this dotted curve is well within the flange material, as previously determined; thus indicating that the cover-plates are of ample length; and showing that the parabolic method of determining their lengths is sufficiently accurate.

In the above discussion the bottom flange only was considered. In accordance with the specification, and with general practice, the top flange will be made the same as the bottom flange, except that the first cover-plate will extend from end to end of the girder. The flanges will not be spliced, as the material for them can be obtained in full lengths.

Rivet-Spacing in Flanges.

The rivets connecting the web-plate with the flange-angles are required to transmit the longitudinal shearing stresses from the web to the flanges. This longitudinal shear per lineal inch, at any section, is assumed equal to the vertical shear at the section divided by the distance, in inches, center to center of gravity of the flanges; and the amount of this shear to be transferred to the flanges, through the rivets, is proportional to

$$\frac{\text{net area of one flange}}{\text{net area of one flange} + \frac{1}{8} \text{ of web-plate section.}}$$

In addition to the longitudinal shear, the top-flange rivets are loaded vertically by the floor- and wheel-concentrations. The weight of floor is 300 lbs, per lin. ft. = 25 lbs. per lin. in.; and the weight of one wheel is 23,000 lbs., which is assumed to be distributed over 36 ins. = 640 lbs. per lin. in. Then the total stress per lineal inch to be resisted by the top-flange rivets is equal to the resultant of the longitudinal shear combined with the vertical load. The dead-load shear at any section is equal to the reaction, or end shear, less the load between this section and the end. The live-load shears will be scaled from the shear-diagram, Fig. 2.

The required pitch of the flange-rivets will be found to increase gradually from the ends towards the center of the girder; and, consequently, to be greater at one end of a panel than at the other end. In the following computations the rivet-pitch is determined at the center of each panel, which is the average pitch required for the whole panel. By this method, the rivets at one end of a panel will be a little too far apart, and those at the other end will be a little too close together; but the total number of rivets in the panel will be correct. There should be no objection to treating the subject in this manner, provided the panel lengths are not greater than the depth of the girder.

Rivet-Spacing in Flanges for Panel a b.—

Vertical Shear at Center of Panel:

Dead-load = 19,500 − (650 × 3)	= 17,500
Live-load (from Fig. 2)	= 97,000
Impact = $\frac{97,000^2}{97,000 + 17,500}$	= 82,100
	196,600 lbs.

Distance, c. to c. of gravity of flanges = 69 ins.

Longitudinal shear per lin. in. at center of gravity of flanges = 196,600 ÷ 69 = 2,840 lbs.

By referring to Fig. 3, it will be found that the flange-section at the center of panel *a b* consists of two 6 x 6 x ⅝-in. angles = 11.72 sq. ins. net area; and ⅛ of a 72 x ⅜-in. web-plate = 3.37 sq. ins.; then

Longitudinal shear per lin. in. on rivets $= 2{,}840 \times \dfrac{11.72}{11.72+3.37}$ = 2,200 lbs.

Bearing value of one ⅞-in. rivet on ⅜-in. plate (by Table II, Chap. I) = 7,220 lbs.

Required spacing of bottom-flange rivets = 7,220 ÷ 2,200 = 3.27 ins.

Vertical Load per Lineal Inch on Top-Flange Rivets:

Dead-load	=	25
Live-load	=	640
Impact = $\dfrac{640^2}{640+25}$	=	615
		1,280 lbs.

Total stress, per lineal inch on top-flange rivets $= \sqrt{2{,}200^2 + 1{,}280^2}$ = 2,550 lbs.

Required spacing of top-flange rivets = 7,220 ÷ 2,550 = 2.83 ins.

Rivet-Spacing in Flanges for Panel b c.

Vertical Shear at Center of Panel:

Dead-load = 19,500 − (650 × 9)	=	13,600
Live-load (from Fig. 2)	=	77,000
Impact = $\dfrac{77{,}000^2}{77{,}000+13{,}600}$	=	65,400
		156,000 lbs.

Distance center to center of gravity of flanges = 70 ins.

Longitudinal shear per lineal inch at center of gravity of flanges = 156,000 ÷ 70 = 2,230 lbs.

By referring to Fig. 3, it will be found that the net flange-section at center of panel *b c* consists of

2 angles 6 x 6 x ⅝-in.	=	11.72
1 plate 15 x ⅝-in.	=	8.13
		19.85 sq. ins.,

and the equivalent flange-area of web-plate, as before, = 3.37 sq. ins.

Then, longitudinal shear per lineal inch on rivets = $2{,}230\times \frac{19.85}{19.85+3.37} = 1{,}900$ lbs.

Required spacing of bottom-flange rivets = $7{,}220 \div 1{,}900$ = 3.8 ins.

Total stress per lineal inch on top-flange rivets $= \sqrt{1{,}900^2+1{,}280^2} = 2{,}290$ lbs.

Required spacing of top-flange rivets = $7{,}220 \div 2{,}290$ = 3.16 ins.

It will be unnecessary, in the present example, to proceed further; for, in accordance with clause 24 of specification, the rivet-pitch in the vertical legs of top-flange-angles shall not exceed 4 ins., and the rivet-pitch in cover-plates shall not exceed 6 ins. If the rivets in vertical legs were spaced 4 ins., those in the cover-plates would also require to be spaced 4 ins. in order to stagger with the former; and this arrangement would require a greater number of rivets than would be the case if the spacing in the vertical legs were 3 ins., and in the flange-plates 6 ins. Sometimes the rivet-spacing in the bottom flange is made greater than that in the top flange, but it is usually considered more economical to make both flanges alike. Consequently, the pitch of rivets in the vertical legs of both top and bottom flange-angles will be made $2\frac{1}{2}$ ins. from a to b, and 3 ins. from b to center of span, as shown in Fig. 4.

Rivet-Spacing in Flange-Plates.—The net area of one flange-plate = 8.13 sq. ins. Then the value of plate in tension = $8.13\times 16{,}000$ = 130,000 lbs. The value of one $\frac{7}{8}$-in. rivet in single shear, as per Table II, Chap. I, = 6,610 lbs. The distance from the end of the first flange-plate to the end of the second flange-plate, as shown in Fig. 3, = 7 ft. = 84 ins.; and the full value of the first flange-plate must be developed, by the connecting rivets, in this distance. Therefore, the number of rivets required in 84 ins. of length = $130{,}000 \div 6{,}610$ = 20; and, since there are two lines of rivets in the plate, the required spacing will be $\frac{84\times 2}{20}$ = 8.4 ins. Since this spacing exceeds that allowed by specification, the pitch of rivets in flange-plates will be 6 ins.

throughout, except in the top flange, panel *a b*, where the pitch will be 5 ins., to stagger with those in vertical legs of angles, as shown in Fig. 4.

WEB-SPLICE.

For an ideal web-splice, the splice-plates should be of the full depth of the web-plate, but this arrangement is not usually practicable. Some engineers use horizontal splice-plates below the top flange-angles and above the bottom flange-angles, which are assumed to resist the bending stresses; and narrow vertical splice-plates between, which are assumed to resist the vertical shearing stresses. This form of splice is objectionable, for the horizontal plates concentrate large stresses in the web-plate, and thereby induce in it undesirable secondary stresses. Another method is to use vertical splice-plates, of a depth equal to the clear distance between the flange-angles, and horizontal flats on the flange-angles. One objection to this is that the rivets in the flange-angles are called on for double duty, as they are also required for the longitudinal shearing stresses; another objection is that the splice-plates on the angles are usually in the way of the stiffeners, thus requiring either additional crimping or additional fillers. Still another method is to splice the web-plate for only a portion of its bending value, assuming its depth equal to the distance between flange-angles; and to arrange to have the splice at some point where the flange material is in excess of that required for the bending moment, viz., near the end of a cover-plate. The only objection to this is that it may be desirable to splice the web-plate at some point where the flange-material is fully utilized, as at the center of a girder.

The most satisfactory form of web-splice, and one which may be used at any point of the girder, consists of two plates, of a depth equal to the distance between flange-angles, and wide enough to contain sufficient rivets to resist the total bending moment attributed to the web-plate. With this splice there will be some slight secondary stresses in the web; but these secondary stresses will be very much less than those induced by the first form of splice mentioned above. The web-splice must be capable of resisting bending, as well as shear; but if capable of resisting the full bending value of the web-plate, it will usually be amply strong for the shear.

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The net flange-section at this point, as shown in Fig. 3, is

$\frac{1}{8}$ of 72 x $\frac{3}{8}$-in. web-plate	=	3.37
2 angles, 6 x 6 x $\frac{5}{8}$-in.	=	11.72
1 plate, 15 x $\frac{5}{8}$-in.	=	8.13
		23.22 sq. ins.;

and the bending moment to be resisted by web-plate = 1,935,000 $\times \frac{3.37}{23.22}$ = 280,800 ft.-lbs. = 3,369,600 in.-lbs.

Stress on outer rivets due to bending = 3,369,600 ÷ 550 = 6,120 lbs., which is less than the permissible stress, as above.

2.—Maximum Shear, with Corresponding Moment (wheel 2 at *c*).

MOMENTS:	Dead-load	=	187,200
	Live-load (not max.)		948,000
	Impact	=	792,300
			1,927,500 ft.-lbs.
SHEARS:	Dead-load	=	11,700
	Live-load (max.)	=	75,000
	Impact	=	65,000
			151,700 lbs.

Stress per rivet due to vertical shear = 151,700 ÷ 48 = 3,160 lbs. Permissible stress on outer rivets for bending = $\sqrt{7,220^2 - 3,160^2}$ = 6,490 lbs.

Bending moment to be resisted by web-plate = 1,927,500 $\times \frac{3.37}{23.22}$ = 280,000 ft.-lbs. = 3,360,000 in.-lbs.

Stress on outer rivets due to bending = 3,360,000 ÷ 550 = 6,110 lbs. Thus the splice satisfies both conditions.

STIFFENERS.

The end stiffeners, which act as columns, receive their load from the web-plate and transmit it to the bearing-plates. As the ratio of their length to their radius of gyration is small, the unit stress of 16,000 lbs. may be used without reduction. The total end shear, including dead-load, live-load, and impact, = 222,000 lbs. Then 222,000 ÷ 16,000 = 13.9 sq. ins. required in stiffeners.

Four angles, 5 x 3½ x $\frac{7}{16}$-in. = 14.12 sq. ins., will be used. Fillers, 3½ x ⅝-in., will be placed between the web-plate and these stiffeners. The number of rivets required = 222,000 ÷ 7,220 = 31. The intermediate stiffeners will, in accordance with specification, be of two angles 4 x 3½ x ⅜-in. These will be offset or crimped to fit over flange-angles.

Bearing-Plates.

Supposing the masonry to be of sound limestone, the permissible pressure per sq. in. is 400 lbs. Then, area of bearing-plates required = 222,000 ÷ 400 = 555 sq. ins. Two plates, 24 x 24 x ⅞-in., will be used. The area of one plate = 576 sq. ins. The upper plates at one end will have slotted holes for the anchor-bolts, to allow for expansion.

Laterals and Brace-Frames.

There will be lateral bracing in the plane of the top flanges only; and brace-frames at the ends to carry the wind force from the laterals to the abutments. There will also be brace-frames at c, e, g, and i, to stiffen the bottom flanges.

The lateral truss shown in Fig. 3 consists of 5 panels, each of 12 ft. Depth of truss = 8 ft Length of diagonals = $\sqrt{8^2+12^2}$ = 14.42 ft. For convenience, the stationary wind force will be called "dead-load," and the moving wind force, "live-load."

Dead-load per lin. ft = 7 × 30 lbs. = 210 lbs.
Live-load per lin. ft. = 300 lbs.
Panel dead-load = 210 × 12 = 2,520 lbs.
Panel live-load = 300 × 12 = 3,600 lbs.
Shear in panel $a\,c$ = (2,520 × 2) + (3,600 × 10/5) = 12,240.
" " " $c\,e$ = (2,520 × 1) + (3,600 × 6/5) = 6,840.
" " " $e\,g$ = 3,600 × 3/5 = 2,160.

$$\text{Stress in 1st diagonal} = 12{,}240 \times \frac{14.42}{8} = 22{,}000.$$

" " 2d " = 6,840 × " = 12,300.
" " 3d " = 2,160 × " = 3,900.

The required sections and sizes used are shown in Fig. 3. The force applied at the top of end brace-frames will be equal to 2½ panels of dead- and live-load = 2½ × (2,520 + 3,600) = 18,360 lbs. One-half of this force is assumed to be resisted by

each diagonal: one in tension, and the other in compression. Stress in top strut = 18,360 x $\frac{1}{2}$ = 9,180. Length of diagonals = $\sqrt{5.5^2 + 8^2}$ = 9.7 ft. Stress in diagonals = 9,180 × $\frac{9.7}{8}$ = ± 11,100. There is no stress in bottom strut. For the top strut, one angle, 4 x 4 x $\frac{5}{16}$-in. = 2.4 sq. ins. will be assumed. The least radius of gyration = 0.82 in., and $\frac{l}{r} = \frac{8}{0.82}$ = 10, which corresponds to a permissible unit stress of 8,890 lbs. for square ends. Then value of strut = 8,890 × 2.4 = 21,300 lbs. The diagonals support one another at the center, so their half length may be used in figuring their capacity. Assuming one angle, 3 x 3 x $\frac{5}{16}$-in. = 1.78 sq. ins., its least radius of gyration = 0.60 in. and $\frac{l}{r} = \frac{4.85}{0.60}$ = 8, which corresponds to a permissible stress per sq. in. of 10,580 lbs. for square ends. Then value of strut = 10,580 × 1.78 = 18,800 lbs. The bottom strut will be like that at the top, viz., one angle 4 x 4 x $\frac{5}{16}$-in. For the intermediate brace-frames, the top and bottom struts will be made of one angle 3½ x 3½ x $\frac{5}{16}$-in. each; and the diagonals, of one angle, 3 x 2½ x 5/16-in.

ESTIMATED WEIGHT.

TWO GIRDERS.

Flanges.—	8 angles, 6x6x$\frac{5}{8}$-in.	@24.2	lbs.	62	ft.	long	= 12,000
"	2 plates, 15x$\frac{5}{8}$-in.	@31.88	"	62	"	"	= 3,950
"	2 plates, 15x$\frac{5}{8}$-in.	@31.88	"	45.75	"	"	= 2,920
"	4 plates, 15x$\frac{5}{8}$-in.	@31.88	"	31.75	"	"	= 4,050
Webs.—	2 plates 72x$\frac{3}{8}$-in.	@91.8	"	62	"	"	= 11,380
Web-splices.—	4 plates, 19x$\frac{3}{8}$-in.	@24.23	"	5	"	"	= 480
Stiffeners.—	16 angles, 5x3½x$\frac{7}{16}$-in.	@12.0	"	6	"	"	= 1,150
"	36 angles, 4x3½x$\frac{3}{8}$-in.	@ 9.1	"	6	"	"	= 1,970
Fillers.—	16 flats, 3½x$\frac{5}{8}$-in.	@ 7.44	"	5	"	"	= 590
Bearings.—	8 plates, 24x$\frac{7}{8}$-in.	@71.4	"	2	"	"	= 1,140
Anchors.—	8 rounds, 1⅛-in. diam.	@ 3.38	"	1.25	"	"	= 40
							39,670
Rivet-heads, (4%)							= 1,580
							lbs. 41,250

LATERALS.

	4 angles, 3x3x$\frac{3}{8}$-in.	@ 7.2	lbs. 12	ft. long	=	350
	6 angles, 3x3x$\frac{5}{16}$-in.	@ 6.1	" 13	" "	=	480
Gussets.—	8 plates, 12x$\frac{5}{16}$-in.	@ 12.75	" 2	" "	=	200
"	12 plates, 10x$\frac{5}{16}$-in.	@ 10.62	" 1.5	" "	=	190
"	2 plates, 20x$\frac{5}{16}$-in.	@ 21.24	" 2	" "	=	80
"	3 plates, 15x$\frac{5}{16}$-in.	@ 15.94	" 1.75	" "	=	80
						1,380
Rivet-heads, (2%)					=	30
						lbs. 1,410

ONE END BRACE-FRAME.

Top and Bottom.—	2 angles, 4x4x$\frac{5}{16}$-in.	@ 8.2	lbs. 7.5	ft. long	=	120
Diagonals .—	2 angles, 3x3x$\frac{5}{16}$-in.	@ 6.1	" 8	" "	=	100
Gussets.—	4 plates, 12x$\frac{5}{16}$-in.	@ 12.75	" 1.25	" "	=	60
"	1 plate, 7x$\frac{5}{16}$-in.	@ 7.44	" 0.75	" "	=	5
						285
Rivet-heads, (2%)					=	5
						lbs. 290

ONE INTERMEDIATE BRACE-FRAME.

Top and Bottom.—	2 angles, 3½x3½x$\frac{5}{16}$-in.	@ 7.1	lbs. 7.75	ft. long	=	110
Diagonals.	2 angles, 3 x2½x$\frac{5}{16}$-in.	@ 5.5	" 8	" "	=	90
Gussets.—	4 plates, 12x$\frac{5}{16}$-in.	@ 12.75	" 1.25	" "	=	60
"	1 plate, 7x$\frac{5}{16}$-in.	@ 7.44	" .75	" "	=	5
						265
Rivet-heads, (2%)					=	5
						lbs. 270

SUMMARY.

2 Girders	=	41,250
Top Laterals	=	1,410
2 End Brace-Frames @ 290 lbs.	=	580
4 Intermediate Brace-Frames @ 270 lbs.	=	1,080
Total	=	44,320 lbs.

Weight per lin. ft. = 44,320 ÷ 62 = 715 lbs., whereas the assumed weight was 700 lbs.

An alternative form of cross-section is shown in Fig. 4*a*. In this the top flange consists of 4 angles 6 x 6 x ½ -in. which extend from end to end of the girder; and these are reinforced at the center of the span by 2 side-plates 11 x ½ -in. Some railway

engineers prefer this construction, because it requires less labor in framing the ties. The chief objection to it is that it is not so economical as the ordinary flange; for with it the effective depth of girder is considerably less than that of the web-plate, and a large proportion of the total flange-section must extend the entire length of the girder. While fairly satisfactory for girders 5 ft. deep or over, it is decidely objectionable for shallower ones, as the lower pair of angles in the top flange, on account of being proportionately so much nearer to the neutral axis of the girder than the upper pair of angles, will be stressed much less.

In order to apply this form of top flange to the girder which has been under discussion, while maintaining the same effective depth, it will be necessary to make the web-plate 6 ins. deeper. In computing the equivalent flange-area of the web-plate, however, it will be assumed that its depth is only equal to the distance from the center of gravity of the top flange to the back of the bottom flange-angles, which is the effective depth of girder. The specification requires that the top flange shall be of the same gross sectional area as the bottom flange. Then,

Gross area of Bottom Flange:

2 angles 6 x 6 x ⅝-in.	=	14.22
2 plates 15 x ⅝-in.	=	19.75
		33.97 sq. ins.

Gross area of Top Flange:

4 angles 6 x 6 x ½-in.	=	23.00
2 plates 11 x ½-in.	=	11.00
		34.00 sq. ins.

CHAPTER III.

THE DESIGN OF A 100-FT. DECK WARREN GIRDER.

The deck Warren girder, designed to carry the ties directly on the top chords, makes a substantial structure. It is most suitable for spans of from 100 ft. to 125 ft. It will be found to serve as an excellent example for illustrating some important problems in bridge design.

The general dimensions of the structure which will be considered in this chapter are as follows:

Length, 100 ft. c to c., of end bearings (= 8 panels of 12 ft. 6 ins. each).

Depth, 12 ft. 6 ins. c. to c. of chords.

Length of diagonals $= \sqrt{12.5^2 + 12.5^2} = 17.67$ ft.

Width, 10 ft. c. to c. of trusses.

Short panel lengths are used in order to avoid excessive bending stresses in the top chords.

Cross-Ties.

The first thing to consider in a bridge of this kind is the size of ties required. The span of the cross-ties is equal to the distance c. to c. of trusses, viz., 10 ft.; and the loads are applied at the rails, which are approximately 5 ft. c. to c. Thus the distance from the points of application of the loads to the center of end supports = 2.5 ft. The maximum wheel-concentration, as per specification, = 30,000 lbs. which is assumed to be distributed over three ties; therefore, the concentrations on each tie = 10,000 lbs. The live-load moment $= 10{,}000 \times 2.5 = 25{,}000$ ft.-lbs. The dead-load moment is so small it may be neglected. Therefore, the impact moment is equal to that of the live-load; and the total moment = 50,000 ft.-lbs. = 600,000 in.-lbs. Assuming the ties to be of Georgia pine or Douglas fir, the permissible fiber-stress = 1,800 lbs. per sq. in. Then, the section modulus required $= 600{,}000 \div 1{,}800 = 333$. Ties 8 ins.

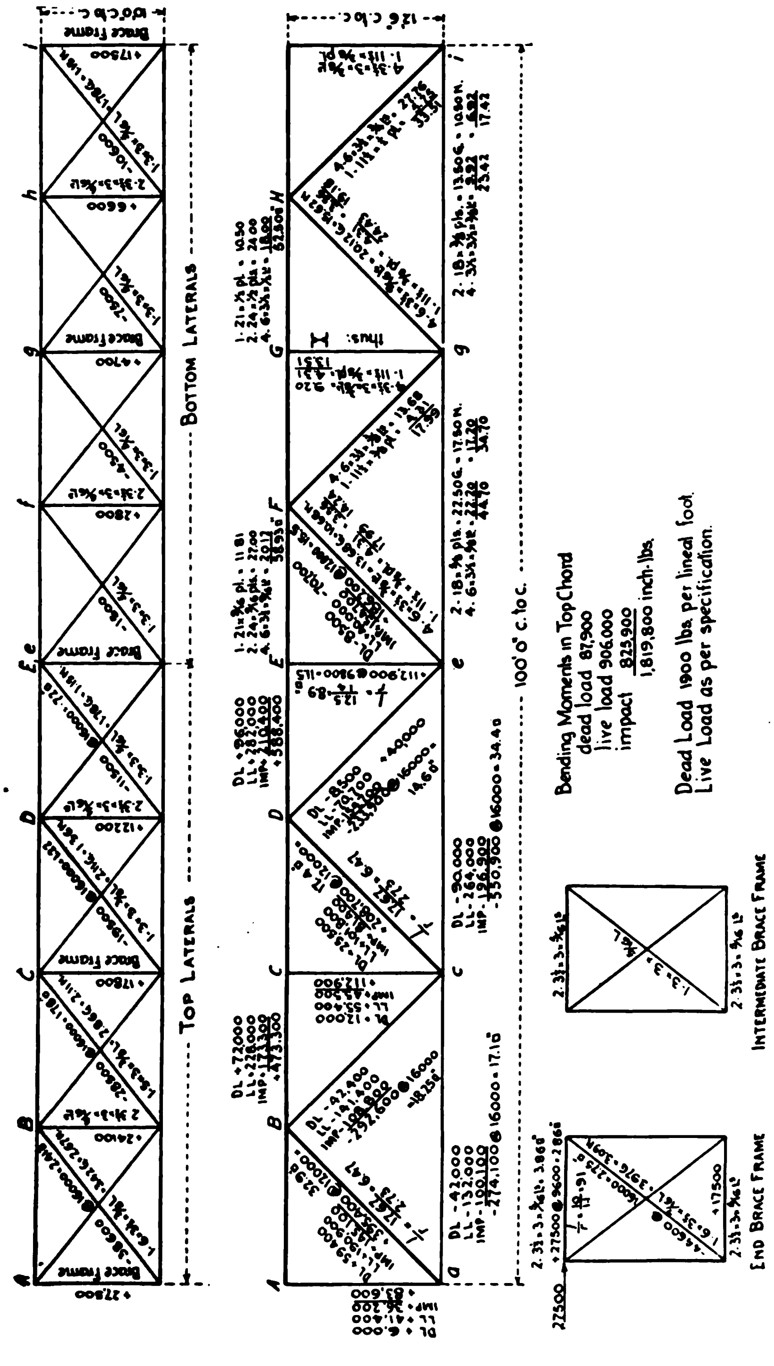


FIG. 5.—100-ft. Deck Warren Girder.

wide and 16 ins. deep will be used. Their section modulus $= \frac{bh^2}{6} = \frac{8 \times 16^2}{6} = 341$.

Loads.

On account of the large size of ties, the assumed weight of floor will be increased from 600 lbs. per lin. ft. to 700 lbs. per lin. ft. By the formula given in Chapter II, the weight of steel per lin. ft. would be $10l + 100 = (10 \times 100) + 100 = 1,100$ lbs. This form of bridge, however, usually runs a little heavier than this; so the weight of steel will be assumed at 1,200 lbs. per lin. ft. Then:

Dead-Load	(floor) =	700
	(steel) =	1,200
	(total) =	1,900 lbs. per lin. ft.

Live-Load as per specification.

Dead-Load Stresses.

It will be found most convenient to compute the dead-load stresses analytically. The panel dead-load for one truss $= \frac{1,900}{2} \times 12.5 = 12,000$ lbs. (about), which is assumed to be concentrated at the upper panel-points. At the end panel-points A and I the concentrations are equal to one-half panel load, $=$ 6,000 lbs. These end-concentrations, however, do not affect the stresses in the truss, so they will be neglected in the following calculations. Since the truss is symmetrical, the reaction at either end, to be used in figuring the stresses, is equal to one-half of the total dead-load, less the half-panel loads at the ends. That is to say, the reaction is equal to $3\frac{1}{2}$ panel loads, $= 12,000 \times 3\frac{1}{2} = 42,000$ lbs. The shear in any panel is equal to this reaction minus any panel-concentrations between the panel considered and the end support and the stress in any diagonal is equal to the shear in the panel in which it is situated, multiplied by the length of diagonal, and divided by the depth of truss, which is equivalent to multiplying by the secant of the angle which the diagonal makes with the vertical. The bending moment at any panel-point is equal to the moment of the reaction about this point, minus the moments of any panel-concentrations between

this point and the end support; and the stress in any chord-section is equal to the moment at the panel-point where the diagonals in the panel intersect the opposite chord, divided by the depth c. to c. of chords. The actual reaction, or load on the masonry, which will be used in designing the pier-members, is equal to 4 panel-loads, $= 12{,}000 \times 4 = 48{,}000$ lbs. The following table of dead-load stresses will now be readily understood:

TABLE OF DEAD-LOAD STRESSES.

Shear in panel AB	$= 42{,}000 - 0$	$= 42{,}000$
" " BC	$= 42{,}000 - 12{,}000$	$= 30{,}000$
" " CD	$= 42{,}000 - (12{,}000 \times 2)$	$= 18{,}000$
" " DE	$= 42{,}000 - (12{,}000 \times 3)$	$= 6{,}000$
Moment at panel-point B	$= 42{,}000 \times 12.5$	$= 525{,}000$
" " D	$= (42{,}000 \times 37.5) - [12{,}000 \times (12.5 + 25)]$	$= 1{,}125{,}000$
" " c	$= (42{,}000 \times 25) - (12{,}000 \times 12.5)$	$= 900{,}000$
" " e	$= (42{,}000 \times 50) - [12{,}000 \times (12.5 + 25 + 37.5)]$	$= 1{,}200{,}000$
Stress in member aB	$= 42{,}000 \times \frac{17.67}{12.5}$	$= +59{,}400$
" " Bc	$= 30{,}000 \times \frac{17.67}{12.5}$	$= -42{,}400$
" " cD	$= 18{,}000 \times \frac{17.67}{12.5}$	$= +25{,}500$
" " De	$= 6{,}000 \times \frac{17.67}{12.5}$	$= -8{,}500$
" " ac	$= 525{,}000 \times \frac{1}{12.5}$	$= -42{,}000$
" " ce	$= 1{,}125{,}000 \times \frac{1}{12.5}$	$= -90{,}000$
" " BD	$= 900{,}000 \times \frac{1}{12.5}$	$= +72{,}000$
" " DF	$= 1{,}200{,}000 \times \frac{1}{12.5}$	$= +96{,}000$
" " Aa	$= 12{,}000 \times \frac{1}{2}$	$= +6{,}000$
" " Cc and Ec	$= 12{,}000 \times 1$	$= +12{,}000$

In accordance with specification, the bending moment in the top chord will be assumed equal to three-fourths of that for a simple span of one panel-length. The dead-load is as follows:

Weight of floor, per lin. ft. = $700 \times \frac{1}{2}$	= 350
Weight of chord-section per lin. ft. (assumed)	= 150
Total dead-load, per lin. ft.	= 500 lbs.

Length of panel = 12.5 ft.

Dead-Load Moment in Top Chord $= \dfrac{500 \times 12.5^2}{8} \times \dfrac{3}{4} = 7{,}325$ ft. lbs. = 87,900 in.-lbs.

Live-Load Stresses.

For the live-load stresses, the moment-diagram, Fig. 1, will be used. A diagram of the truss should be constructed, to the same scale, on tracing cloth, and placed on the moment-diagram in the various positions giving the maximum reaction, shears, and moments, as required.

Maximum Reaction at a.—Since every load on the span contributes to the reaction at a, then, in order to obtain the maximum reaction, the heavier loads should be placed as near this point as possible. Consequently, the maximum reaction occurs with the first driving-wheel (wheel 2) at a. Then, placing the diagram of truss on the moment-diagram in this position, verticals are dropped from a and i, intersecting the equilibrium-polygon, and the closing line drawn through these intersections. A line is then drawn from the point O in force-polygon, parallel with the closing line, and intersecting the vertical load-line. The reaction a is equal to the total load above this intersection, (less load 1 which is not on the span) = 164,000 lbs.

Position of Load for Maximum Shear.—For a truss with parallel chords, the maximum shear in any panel (with reference to the left-hand support) occurs with a wheel near the head of the train placed at the adjacent right-hand panel-point, when the load in the panel to the left of this point is equal to or less than the total load on span divided by the number of panels in the span. In other words, the average load per foot in the panel (not including the wheel-load at the adjacent right-hand panel-point), should be equal to or less than the average load per foot on the whole span. The proof of this may be found in Johnson's "Theory and Practice of Modern Framed Structures."

For the maximum shear in panel A B, the diagram of truss is placed on the moment-diagram with wheel 3 at B, as shown in Fig. 6. In this position, the load in the panel to the left of B = 34,500 − 11,500 = 23,000 lbs.; and the total load on span =

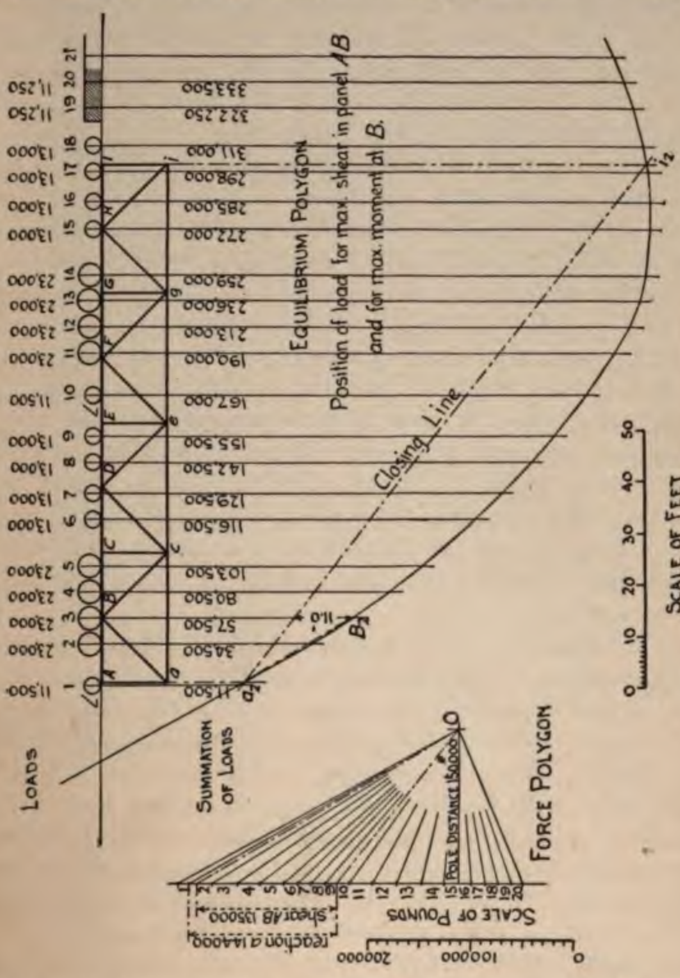


FIG. 6.—100-ft. Deck Warren Girder.

298,000 − 11,500 = 286,500 lbs.; which figures are obtained from the summation of loads on moment-diagram. Then 23,000 lbs. is less than 286,500 ÷ 8 = 35,800 lbs. That is to say, the load in the panel to the left of B is less than the total load on span, divided by the number of panels in span. With wheel 4 at B,

the load in the panel to the left of this point would be 57,500 − 11,500 = 46,000 lbs.; and the total load on span, 311,000 − 11,500 = 299,500 lbs. Since 46,000 is greater than 299,500 ÷ 8 = 37,400, the maximum shear does not occur with wheel 4 at B, but with wheel 3 at this point, as shown. Verticals are dropped from a and i, intersecting the equilibrium-polygon in a_2 and i_2, and the closing line drawn through these points. Then a line drawn through the point O in force-polygon, parallel with the closing line $a_2\, i_2$, and intersecting the vertical load-line, determines the reaction for this position of load. It is equal to the total load above this last intersection, less wheel 1 (which is off the span), = 144,000 lbs., as shown. This reaction is greater than the shear in panel $A\ B$, for a portion of wheel 2 is carried to the vertical end-post by the top chord, which acts as a stringer. The part of this load which goes to the end-post is obtained by dropping a vertical from panel-point B to the equilibrium-polygon, intersecting it in the point B_2; and then drawing a line through the point O in force-polygon, parallel with a line through $a_2\, B_2$, and intersecting the vertical load-line. The part of the load-line above this last intersection (neglecting load 1 which is off the span) represents the portion of wheel 2 which is carried by the end-post; and the distance between the two intersections on load-line represents the shear in panel $A\ B$, which is 135,000 lbs., as shown in Fig. 6.

For the maximum shear in panel $B\ C$, the diagram of truss is placed on moment-diagram with wheel 2 at C as shown in Fig. 7. Now the load in panel to the left of C = 11,500 lbs., and the total load on span = 259,000 lbs. Then 11,500 is less than 259,500 ÷ 8 = 32,400. With wheel 3 at C, the load to the left of this point would be 34,500 lbs.; and the total load on span, 272,000 lbs. Then 34,500 is greater than 272,000 ÷ 8 = 34,000; and, consequently this position does not give the maximum shear; but it occurs with wheel 2 at C. In this position, verticals are drawn through a and i, intersecting the equilibrium-polygon in a_2 and i_2; and the closing line drawn through these points. Then a line drawn through the point O in force-polygon, parallel with closing line, and intersecting the vertical load-line, determines the reaction a, which is equal to the portion of the load-line above this intersection, = 105,000 lbs. The shear in panel $B\ C$ is equal to this reaction at a, less the portion of wheel 1 which is

carried to panel-point B through the top chord. The portion of this load which goes to panel-point B is determined by dropping verticals through B and C to the equilibrium-polygon, intersecting it in points B_2 and C_2; and then drawing a line through the point O in force-polygon, parallel with a line through the points B_2 and C_2, and intersecting the vertical load-line. The

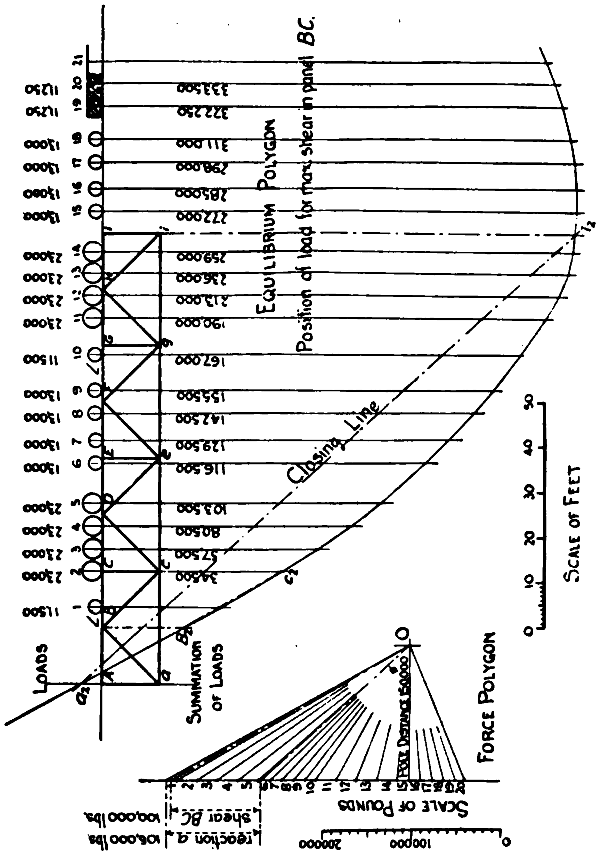


FIG. 7.—100-ft. Deck Warren Girder.

part of the load-line above this last intersection represents the portion of wheel 1 which goes to panel-point B; and the distance between the two intersections on the load-line represents the shear in panel B C, which is 100,000 lbs., as shown.

The maximum shear in panel C D, which occurs with wheel 2 at D, = 72,000 lbs.

The maximum shear in panel D E, which occurs with wheel 2 at E, = 50,000 lbs.

The maximum shear in panel E F, which occurs with wheel 2 at F, = 28 500 lbs.

Position of Load for Maximum Moment.—For the maximum moment at any panel-point, the load should cover the span, while the heavier wheel-loads should be brought near the point. Then, the total load to the left (not including the load at the panel-point in question) divided by the number of panels between this point and the left-hand support, should be equal to or less than the total load on span divided by the number of panels in the span. In other words, the load per foot (or per panel) to the left of the point (not including the load at the point) should be equal to or less than the load per foot (or per panel) on the whole span. The proof of this may be found in Johnson's "Theory and Practice of Modern Framed Structures." There may be two or three positions of the load which will satisfy the above conditions, in which case all should be tried to ascertain the greatest moment. With a little practice, however, the correct position of the load can usually be judged by inspection.

For the maximum moment at B, which is the point of moments for obtaining the stress in a c, wheel 3 is placed at this point, as shown in Fig. 6. The load to the left of B = 23,000 lbs., and the total load on span = 286,500 lbs. Then $23{,}000 \div 1 = 23{,}000$ is less than $286{,}500 \div 8 = 35{,}800$. With wheel 4 at B, it would be found that the load to the left of this point, divided by 1, would be greater than the total load on span, divided by 8. Thus the position shown is the correct one. It should be noted that the position of load for the maximum shear in the end panel, and for the maximum moment at the first panel-point from the end, is always the same. In Fig. 6, the ordinate between the closing line and the equilibrium-polygon, at B, measures 11 ft. Then the moment at this point = $150{,}000 \times 11$ = 1,650,000 ft.-lbs.

For the maximum moment at c, which is the point of moments for the stress in B D, wheel 4 is placed at this point. The total load to the left of C = 57,500, and the total load on span = 272,000. Then $57{,}500 \div 2 = 28{,}750$ is less than $272{,}000 \div 8$ = 34,000. The ordinate between the closing line and the equilibrium polygon, vertically below c, measures 19 ft.; and the moment at this point = $150{,}000 \times 19$ = 2,850,000 ft.-lbs.

For the maximum moment at D, which is the point of moments for the stress in $c\,e$, wheel 6 is placed at this point. The total load to the left of D = 103,500, and the total load on span = 285,000. Then $103{,}500 \div 3 = 34{,}500$ is less than $285{,}000 \div 8 = 35{,}600$. The ordinate at this point measures 22 ft.; and the moment = $150{,}000 \times 22 = 3{,}300{,}000$ ft.-lbs. Practically the same result would be obtained with wheel 12 or wheel 13 at this point.

For the maximum moment at e, which is the point of moments for the stress in $D\,F$, wheel 13 is placed at this point. The total load to the left of E (not including wheels 1 to 5, which are off the span) = 109,500; and the total load on span = 241,250. Then $109{,}500 \div 4 = 27{,}400$ is less than $241{,}250 \div 8 = 30{,}100$. The ordinate at this point measures 23.5 ft.; and the moment = $150{,}000 \times 23.5 = 3{,}525{,}000$ ft.-lbs.

TABLE OF LIVE-LOAD STRESSES.

Shear in panel	$A\,B$ =	135,000
" "	$B\,C$ =	100,000
" "	$C\,D$ =	72,000
" "	$D\,E$ =	50,000
" "	$E\,F$ =	28,500
Moment at	B =	1,650,000
" "	c =	2,850,000
" "	D =	3,300,000
" "	e =	3,525,000

Stress in $a\,B$ =	$135{,}000 \times \frac{17.67}{12.5}$	=	+190,900
" " $B\,c$ =	$100{,}000 \times$ "	=	—141,400
" " $c\,D$ =	$72{,}000 \times$ "	=	+101,800
" " $D\,e$ =	$50{,}000 \times$ "	=	— 70,700
" " $e\,F$ =	$30{,}000 \times$ "	=	+ 40,000
" " $a\,c$ =	$1{,}650{,}000 \times \frac{1}{12.5}$	=	—132,000
" " $B\,D$ =	$2{,}850{,}000 \times$ "	=	+228,000
" " $c\,e$ =	$3{,}300{,}000 \times$ "	=	—264,000
" " $D\,F$ =	$3{,}525{,}000 \times$ "	=	+282,000

With a driving-wheel at A and two driving-wheels in the panel $A\,B$, the maximum stress in $A\,a$ is obtained by taking

moments of these loads about B, and dividing by the length of panel, as follows:

Stress in $A\ a = [23{,}000 \times (2.5+7.5+12.5)] \div 12.5 = 41{,}400$.

With the second or third driving-wheel at one of the intermediate vertical posts, two driving-wheels in one adjacent panel and one in the other, the maximum stress in post is obtained by taking moments of these loads about the opposite end of panels, and dividing by length of panels, as before.

Stress in $C\ c$ and $E\ e = [23{,}000 \times (7.5+12.5+7.5+2.5)] \div 12.5 = 55{,}400$.

For the bending moment in top chord, three driving-wheels should be placed in a panel, with one of them at the center. Then, considered as a simple span, the reaction at either end would be equal to $23{,}000 \times 1\frac{1}{2} = 34{,}500$ lbs.; and the moment at center of panel would be equal to $(34{,}500 \times 6.25) - (23{,}000 \times 5) = 100{,}625$ ft.-lbs. The moment for a continuous chord, according to specification, will now be assumed equal to $100{,}625 \times \frac{3}{4} = 75{,}500$ ft.-lbs. $= 906{,}000$ in.-lbs.

The dead- and live-load stresses are summarized on the stress-diagram, Fig. 5, with impact stresses added according to specification. Taking member $a\ B$ as an example:

$$\begin{aligned} &\text{Dead-load} && = +\ 59{,}400 \\ &\text{Live-load} && = +190{,}900 \\ &\text{Impact} = \frac{190{,}900^2}{190{,}900+59{,}400} && = +\ 145{,}100 \\ & && \overline{+\ 395{,}400} \text{ lbs.} \end{aligned}$$

In member $D\ e$, which corresponds with member $e\ F$, the dead-load stress $= -8{,}500$, the maximum live-load tension $= -70{,}700$, and the maximum live-load compression $= +40{,}000$. The impact stress is then equal to

$$\frac{(70{,}700+40{,}000)^2}{70{,}700+8{,}500} = \pm\ 154{,}700 \text{ lbs.}$$

These stresses should be summarized, as follows:

Dead-load	$-\ 8{,}500$	Dead-load	$-\ 8{,}500$
Live-load	$-\ 70{,}700$	Live-load	$+\ 40{,}000$
Impact	$-\ 154{,}700$	Impact	$+\ 154{,}700$
	$-233{,}900$ lbs.		$+186{,}200$ lbs.

PROPORTIONING OF TRUSS MEMBERS.

The tension members are all proportioned for a unit stress of 16,000 lbs. per sq. in. of net section; and the compression members, for 16,000 lbs. per sq. in., reduced by the formula for fixed ends. For the top chords, however, the unit stress will not require to be reduced, as the panel lengths are less than 36 times the least radius of gyration of these members.

In member a B the total compression = 395,400 lbs., and its length = 17.67 ft. Assuming a section, in the form of a girder, composed of

4 flange-angles,	6 x 4 x $\frac{3}{4}$-in. =	27.76	(4″ legs riveted to web-plate)
1 web-plate	11½ x ½-in. =	5.75	
		33.51 sq. ins.,	

its least radius of gyration, which is about the axis parallel with web-plate, will be computed as follows: The radius of gyration of 2 angles 6 x 4 x $\frac{3}{4}$-in. (with the 4-in. legs back to back, but separated ½-in.) may be found in the Carnegie, or other structural steel handbooks. That about the axis parallel with the 4-in. legs = 3 ins.; and for 4 angles similarly arranged it is the same. The moment of inertia of the four angles is equal to their area multiplied by the square of this radius of gyration, = 27.76 $\times$ $3^2 = 249.84$; but the moment of inertia of the web-plate about this axis is so small that it may be neglected. Then, since $I = A r^2$ (in which I = the moment of inertia, A = the area of section, and r = the radius of gyration), $r = \sqrt{\frac{I}{A}} = \sqrt{\frac{249.84}{33.51}} =$ 2.73 ins. In this calculation it should be noted that the moment of inertia is divided by the total area of section, including that of the web-plate. Now $\frac{l}{r} = \frac{17.67}{2.73} = 6.47$, which (by Table I, Chap. I) corresponds with a unit stress of 12,000 lbs. per sq. in.; and 395,400 ÷ 12,000 = 32.9 sq. ins. required. Consequently, the assumed section is about right.

In member B c, the total tension = 292,600 lbs. Then, 292,600 ÷ 16,000 = 18.25 sq. ins. required. The following section is provided:

	Gross.	Net.
4 angles, 6 x 3½ x $\frac{9}{16}$-in.	= 20.12 sq. ins. =	15.62 sq. ins
1 plate, 11½ x ⅜-in.	= 4.31 " " =	3.56 " "
Total,	24.43 " "	19.18 " "

(Four 1-in. holes are allowed for in the angles, and two in the plate).

In member *c D*, the total compression = 208,700 lbs. The length of member, radius of gyration and, consequently, the permissible unit stress are the same as for *a B*. Then 208,700 ÷ 12,000 = 17.4 sq. ins. required. The following section is provided:

4 angles, 6 x 3½ x ⅜-in.	= 13.68
1 plate, 11½ x ⅜-in.	= 4.31
	17.99 sq. ins.

In member *D e*, or *e F*, the total tension = 233,900 lbs. and the total compression = 186,200 lbs. Then, 233, 900 ÷ 16,000 = 14.6 sq. ins. net area required; and 186,200 ÷ 12,000 = 15.5 sq. ins. gross area required. Thus the tensile stress determines the section used in this case, which is the same as provided for member *c D*.

In the intermediate vertical posts, the total compression = 112,900 lbs., and their length = 12.5 ft. Then, assuming a section composed of

4 flange-angles,	3½ x 3 x ⅜-in. = 9.20	(3-in. legs riveted to web-plate)
1 web-plate,	11½ x ⅜-in. = 4.31	
	13.51 sq. ins.,	

its least radius of gyration, which is about the axis parallel with web-plate, will be computed as follows: The radius of gyration of the angles alone, (from Carnegie, as before,) = 1.7 ins., and their moment of inertia = 9.2 × 1.7² = 26.6. The moment of inertia of the web-plate will be neglected, as before. Then $r = \sqrt{\frac{26.6}{13.51}} = 1.4$ ins. Now, $\frac{l1}{r} = \frac{12.5}{1.4} = 8.9$, which corresponds with a unit stress of 9,800 lbs. per sq. in.; and 112,900 ÷ 9,800 = 11.5 sq. ins. required. The assumed section is a little greater than required for the stress; but, if smaller angles were used, the

radius of gyration would be too small to satisfy the condition that the length of a member shall not exceed 100 times its least radius of gyration. Even with the assumed section, this condition is slightly exceeded. The same section is also used for the vertical end-posts.

In the bottom-chord member $a\ c$, the total tension = 274,100 lbs.; and 274,100 ÷ 16,000 = 17.1 sq. ins. required. The section used is as follows:

	Gross.	Net.
2 web-plates, 18 x ⅜-in.	= 13.50 sq. ins.	= 10.50 sq. ins.
4 flange-angles, 3½ x 3½ x ⅜-in.	= 9.92 " "	= 6.92 " "
Total,	23.42 " "	17.42 " "

(There are four 1-in. holes in each plate and two in each angle).

In the bottom-chord member $c\ e$, the total tension = 550,900 lbs.; and 550,900 ÷ 16,000 = 34.4 sq. ins. required. The section used is as follows:

	Gross.	Net.
2 web-plates, 18 x ⅝-in.	= 22.50 sq. ins.	= 17.50 sq. ins.
4 flange-angles, 6 x 3½ x ⅝-in.	= 22.20 " "	= 17.20 " "
Total,	44.70 " "	34.70 " "

(There are four 1-in. holes in each plate, and two in each angle).

The top chord must be so designed that the maximum fiber-stress due to the combination of direct compression and bending moments shall not exceed 16,000 lbs. per sq. in. The area required for the direct stress is found directly by dividing this stress by 16,000; but, in order to determine the area required for the bending moments, some definite section must first be assumed, its radius of gyration computed, also the distance from the horizontal axis through the center of gravity to the extreme outer fiber, whether top or bottom; for the specification requires that the bending moment at panel-points be assumed equal but opposite to that at the center of panel. The formula for determining the area required for bending will now be developed.

$$M = S f \qquad (1)$$

$$S = \frac{I}{n} \qquad (2)$$

$$I = A r^2 \qquad (3)$$

in which M = bending moment in inch-pounds.
S = section modulus.
n = distance from horizontal axis through center of gravity to extreme outer fibers.
r = radius of gyration about horizontal axis through center of gravity.
A = area of section required for bending.
f = maximum fiber stress.

Substituting Equation (3) in Equation (2), $S = \dfrac{A\,r^2}{n}$ (4)

Substituting Equation (4) in Equation (1), $M = \dfrac{A\,r^2}{n} f$ (5)

The required formula, which is obtained by reducing Equation (5), is

$$A = \frac{M\,n}{r^2 f} \tag{6}$$

In the top-chord member $B\ D$, the total compression = 473,300 lbs.; and the total moment = 1,819,800 in.-lbs. Then, assuming the following section,

1 cover-plate,	21 x ⅜-in.	=	7.87
2 web-plates,	18 x ⅜-in.	=	13.50
4 flange-angles,	6 x 3½ x ⅜-in.	=	13.68
			35.05 sq. ins.,

the radius of gyration* about the horizontal axis is found to be 6.74 ins.; and the distance from this axis to the extreme outer fibers = 11.07 ins.

Area required for direct compression $= \dfrac{473{,}300}{16{,}000} =$ 29.58

Area required for bending moment $= \dfrac{1{,}819{,}800 \times 11.07}{6.74^2 \times 16{,}000} =$ 27.70

sq. ins. 57.28

*For methods of computing the position of center of gravity, moment of inertia, and radius of gyration for a section of this form, see Thomson's "Bridge and Structural Design," page 18.

The assumed section might be increased sufficiently to suit this case; but it would be uneconomical to do so, as the area required for the bending moment is proportionately very large, so a deeper section of the same form will be tried, as follows:

1 cover-plate, 21 x $\frac{3}{8}$-in.	=	7.87
2 web-plates, 24 x $\frac{3}{8}$-in.	=	18.00
4 flange-angles, 6 x $3\frac{1}{2}$ x $\frac{3}{8}$-in.	=	13.68
		39.55 sq. ins.

The radius of gyration about the horizontal axis through the center of gravity is found to be 9.01 ins.; and the distance from this axis to the extreme outer fibers, 14.42 ins. Then,

$$\text{area required for direct compression} = \frac{473{,}300}{16{,}000} = 29.58$$

$$\text{area required for bending moment} = \frac{1{,}819{,}800 \times 14.42}{9.01^2 \times 16{,}000} = 20.20$$

sq. ins. 49.78

The following section is used:

1 cover-plate, 21 x $\frac{1}{2}$-in.	=	10.50
2 web-plates, 24 x $\frac{1}{2}$-in.	=	24.00
4 flange-angles, 6 x $3\frac{1}{2}$ x $\frac{1}{2}$-in.	=	18.00
		52.50 sq. ins.

In the top-chord member *D F*, the total compression = 588,400 lbs.; and the total moment, 1,919,800 in.-lbs. Then,

$$\text{area for direct compression} = \frac{588{,}400}{16{,}000} = 36.77$$

$$\text{area required for bending moment, as before,} = 20.20$$

56.97 sq. ins.

The following section is used:

1 cover-plate, 21 x $\frac{9}{16}$-in.	=	11.81
2 web-plates, 24 x $\frac{9}{16}$-in.	=	27.00
4 flange-angles, 6 x $3\frac{1}{2}$ x $\frac{9}{16}$-in.	=	20.12
		58.93 sq. ins.

The sections used for the various members are given on the right-hand end of stress-diagram, Fig. 5, to avoid crowding.

LATERALS.

The top laterals are designed for a fixed horizontal force of 150 lbs. per lin. ft., plus a moving horizontal force of 400 lbs. per lin. ft. The bottom laterals are designed for a fixed horizontal force of 150 lbs. per lin. ft. only. For convenience, the fixed horizontal force will be called dead-load, and the moving horizontal force, live-load.

Panel dead-load for top and bottom laterals $= 150 \times 12.5 =$ 1,875 lbs.

Panel live-load for top laterals only $= 400 \times 12.5 = 5{,}000$ lbs.

Length of diagonals $= \sqrt{10^2 + 12.5^2} = 16$ ft.

TABLE OF TOP-LATERAL STRESSES.

Shear in panels:

$$AB = (1{,}875 \times 3\tfrac{1}{2}) + (5{,}000 \times \frac{28}{8}) = 24{,}100$$

$$BC = (1{,}875 \times 2\tfrac{1}{2}) + (5{,}000 \times \frac{21}{8}) = 17{,}800$$

$$CD = (1{,}875 \times 1\tfrac{1}{2}) + (5{,}000 \times \frac{15}{8}) = 12{,}200$$

$$DE = (1{,}875 \times \tfrac{1}{2}) + (5{,}000 \times \frac{10}{8}) = 7{,}200$$

Stress in diagonals:

$$\begin{aligned}
\text{1st} &= 24{,}100 \times \frac{16}{10} = -38{,}600 \\
\text{2d} &= 17{,}800 \times \text{"} = -28{,}500 \\
\text{3d} &= 12{,}200 \times \text{"} = -19{,}500 \\
\text{4th} &= 7{,}200 \times \text{"} = -11{,}500
\end{aligned}$$

TABLE OF BOTTOM-LATERAL STRESSES.

Shear in panels:

$$\begin{aligned}
ab &= 1{,}875 \times 3\tfrac{1}{2} = 6{,}600 \\
bc &= 1{,}875 \times 2\tfrac{1}{2} = 4{,}700 \\
cd &= 1{,}875 \times 1\tfrac{1}{2} = 2{,}800 \\
de &= 1{,}875 \times \tfrac{1}{2} = 900
\end{aligned}$$

Stress in diagonals:

$$
\begin{aligned}
\text{1st} &= 6{,}600 \times \frac{16}{10} = -\ 10{,}600 \\
\text{2d} &= 4{,}700 \times \text{“} = -\ \ 7{,}500 \\
\text{3d} &= 2{,}800 \times \text{“} = -\ \ 4{,}500 \\
\text{4th} &= \ \ \ 900 \times \text{“} = -\ \ 1{,}500
\end{aligned}
$$

The stresses are shown on diagram, Fig. 5; also the sections required, and the sizes used. For determining the net areas, two holes, 1 in. in diameter, are assumed in each angle.

END BRACE-FRAMES.

The end brace-frames are designed to resist four panels of dead-load and four panels of live-load, $= 4 \times (1{,}875 + 5{,}000) =$ 27,500 lbs., applied at the top chord, as shown in Fig. 5. The diagonals are assumed to be capable of resisting tension only; so each one is designed for the whole load. Length of diagonals $= \sqrt{10^2 + 12.5^2} = 16$ ft. Stress in diagonals $= 27{,}500 \times \frac{16}{10} =$ $-44{,}000$ lbs. The stress in top strut is equal to the applied load $= +27{,}500$ lbs. One-half of the wind force on top and bottom chords is assumed to be resisted by the anchor-bolts of the windward truss, and the other half is assumed to be transmitted to those of the leeward truss through the bottom struts. Thus the stress in these members is equal to $4 \times (1{,}875 + 1{,}875 + 5{,}000) \times \frac{1}{2} = +17{,}500$ lbs.

DETAILS.

Fig. 8 is a detail drawing showing an inside view of one-half of a truss, one-quarter plan of top laterals, one-quarter plan of bottom laterals, one-half of an end brace-frame, one-half of an intermediate brace-frame, pier-members, etc. The trusses are supposed to be shipped riveted up; thus all rivets, except those in the connections of laterals and brace-frames, are shop-driven. The rivets used throughout are $\frac{7}{8}$-in. and, since all those in the main connections are in single shear, their value, (as per Table II, Chap. I), $= 6{,}610$ lbs. each; and the number of rivets required in the connection at either end of a member is equal to the total stress in that member divided by the value of one rivet. Thus, for member *a B*, the number of rivets required $= 395{,}400 \div 6{,}610 = 60$.

Bottom-Chord Splice.—The strength of the bottom-chord splice should be equal to or greater than the tensile strength of the member spliced; and care should be taken to insure that each angle and plate composing the member is spliced to its full value. The splice is located in the end panel *a c*, near panel-point *c*, as shown. The gusset-plates here, which are ½-in. thick, are utilized to splice the 18 x ⅜-in. web-plates at a point 1 ft. 7½ ins. distant from the panel-point; and they are also used to splice the vertical legs of the 3½ x 3½ x ⅜-in. angles, at a point 12 ins. from the web-splice. The horizontal legs of the top flange-angles are spliced by 3½ x ½-in. flats, 1 ft. 9 ins. long; and the horizontal legs of the bottom flange-angles are spliced by a 21 x ⅜-in. tie-plate, 1 ft. 9 ins. long. Thus the splice material is considerably in excess of the material spliced. The net area of the four 3½ x 3½ x ⅜-in. angles = 6.92 sq. ins.; and their value in tension = 6.92 × 16,000 = 110,700 lbs. Then, 110,700 ÷ 6,610 = 17 rivets required in these angles, whereas there are 24 rivets provided—12 rivets in the vertical legs and 12 rivets in the horizontal legs. The net area of the two 18 x ⅜-in. web-plates = 10.5 sq. ins.; and their value in tension = 10.5 × 16,000 = 168,000 lbs. Then, 168,000 ÷ 6,610 = 25 rivets required in these plates, whereas there are 32 rivets provided, not counting those in the 3½ x 3½ x ⅜-in. angles. The value of the rivets connecting the bottom-chord member *c e* at panel-point *c* should also be considered. The stress in this member, as shown on stress-diagram, Fig. 5, = 550,900 lbs. There are 98 rivets in the 18 x ⅜-in. web-plates and the vertical legs of the 6 x 3½ x ⅜-in. angles, 16 rivets in the vertical legs of these angles alone, and 12 rivets in the horizontal legs of the angles, making 126 rivets in single shear, which, at 6,610 lbs. = 832,800 lbs. Therefore the splice is amply strong in every respect.

If for any reason it were found necessary to ship the chords and web-members separately, the number of rivets to be field-driven should be increased 25% to allow for inferior riveting.

Top-Chord Splice.—The top chord is spliced in panel *C D*, 1 ft. 10½ ins. from panel-point *D*. The joint is faced to make a perfect bearing for the abutting members, in order that the direct compression may be transmitted directly from one section to the other. The web and flange splice-plates are required only to hold the sections in line, and to provide for the local bending and shearing stresses in the chord.

Gusset-Plates.—The size of the gusset-plates is determined by the number of rivets required in the connections; but their thickness is largely a matter of judgment, and partly governed by the thickness of web-plates in top and bottom chords, as will be shown presently. The width of the cover-plate on top chord is 21 ins., and the width of the flange-angles is $3\frac{1}{2}$ ins. Then, keeping the edge of the flange-angles flush with the edges of the cover-plate, the distance back to back of angles, (or out to out of web-plates), $= 21-(3\frac{1}{2}+3\frac{1}{2}) = 14$ ins., which is constant throughout for both top and bottom chords. Beginning at panel-point *B*, where the web-plates are $\frac{1}{2}$-in. thick, and making the gusset-plates $\frac{5}{8}$-in. thick, the clear distance between the gusset-plates $= 14-(\frac{1}{2}\times 2)-(\frac{5}{8}\times 2) = 11\frac{3}{4}$ ins. Keeping this clear distance throughout, in order to make all web-members the same width, the thickness of gusset-plates at the various panel-points is determined as follows: At panel-point *B*, the thickness of one web-plate, plus one gusset-plate $= \frac{1}{2}$-in. $+\frac{5}{8}$-in. $= 1\frac{1}{8}$ ins. At panel-point *D*, the thickness of web-plates $= \frac{9}{16}$-in.; therefore, the thickness of gusset-plates $= 1\frac{1}{8}-\frac{9}{16} = \frac{9}{16}$-in. At panel-point *a*, the thickness of web-plates $= \frac{3}{8}$-in.; and the thickness of gusset-plates $= 1\frac{1}{8}-\frac{3}{8} = \frac{3}{4}$-in. At panel-points *c* and *e* the web-plates are $\frac{5}{8}$-in. thick; then, the thickness of gusset-plates $= 1\frac{1}{8}-\frac{5}{8} = \frac{1}{2}$-in. The width of the web-members is $\frac{1}{8}$-in. less than the clear distance between gusset-plates. This is to facilitate the assembling of the trusses.

Pier Members.—As found previously, the dead-load reaction = 48,000 lbs., and the maximum live-load reaction = 164,000 lbs. Then, the total reaction, including impact is as follows:

$$
\begin{array}{lr}
\text{Dead-load} & = 48{,}000 \\
\text{Live-load} & = 164{,}000 \\
\text{Impact} = \dfrac{164{,}000^2}{164{,}000+48{,}000} & = 127{,}000 \\
\hline
 & 339{,}000 \text{ lbs.}
\end{array}
$$

Assuming the bridge-seat to be of sound limestone, the permissible pressure per square inch = 400 lbs. Then, $339{,}000 \div 400 = 848$ sq. ins. required in bed-plate. The bed-plates provided are 33×33 ins. $= 1{,}089$ sq. ins. Assuming rollers 5 ins. in diameter, the permissible pressure on them, per lineal inch, =

$1,200\sqrt{5} = 2,688$ lbs. Then, $339,000 \div 2,688 = 126$ lin. ins. of rollers required. There are 5 rollers provided, having a bearing length of 25 ins. each, = 125 lin. ins. in all. The ends of the rollers are turned down to 1½ ins. in diameter, to pass through holes in the 4½ x ½-in. spacing-bars; and cotter-pins are inserted in the ends of the outer rollers to prevent the bars from dropping off. The ends of the intermediate rollers are just long enough to pass through the spacing-bars. The shoe- and bed-plates at the roller end, which are both 1⅝ ins. thick, are planed to 1½ ins. for a width of 25⅜ ins., leaving a ridge ⅛-in. high at each side to keep the rollers in place. The shoe-plate at the fixed end is planed all over to bear on the cast-iron bed-plate, which latter is made of 1¼-in. metal, and faced top and bottom, as shown. The height of the cast bed-plate is made equal to the diameter of the rollers plus the thickness of the roller bed-plate, thus enabling the masonry at both ends of the span to be built at the same elevation.

Sway and Lateral Bracing.—As the rivets in the connections of the sway and lateral bracing are field-driven, their number is increased by 25% over the number required for shop-driven rivets. The top and bottom lateral connection-plates are intended to be in direct contact with the flange-angles, as shown; and, wherever tie-plates or lattice-bars interfere, the latter are to be sprung to allow the lateral plates to be placed between them and the flange-angles.

Camber.—The trusses are cambered, or slightly arched, in order to offset the deflection due to dead- and live-loads. This is done by adding ⅛-in. to each top-chord panel, except the end ones. The end panels, however, are shortened ¼-in. each to keep the posts over the abutments more nearly vertical.

Estimated Weight.

The weight of the structure will now be estimated from the detailed drawings, as follows:

Top Chords. TWO TRUSSES.

16 angles	6 x3½ x ½-in.	@15.30 "	36 " 10½	ins. long	= 9,030
4 plates	21 x ½ "	@39.10 lbs.	36 ft. 10½	" "	= 5,770
8 plates	24 x ½ "	@40.80 "	36 " 10½	" "	= 12,040
2 plates	21 x $\frac{9}{16}$ "	@40.17 "	28 ft. 9¼	" "	= 2,310
				Carried forward,	29,150

				Brought forward,			29,150
4 plates	24 x $\frac{9}{16}$	-in.@45.90 lbs.	28 ft.	$9\frac{1}{4}$ ins.	long	=	5,280
8 angles	6 x$3\frac{1}{2}$x$\frac{9}{16}$	" @17.10 "	28 "	$9\frac{1}{4}$ "	"	=	3,940
4 plates	21 x $\frac{3}{8}$	" @26.78 "	2 "	3 "	"	=	240
16 plates	21 x $\frac{3}{8}$	" @26.78 "	1 "	9	"	=	750
16 plates	21 x $\frac{3}{8}$	" @26.78 "	1 "		"	=	430
4 flats	$3\frac{1}{2}$x $\frac{3}{8}$	" @ 4.47 "		$10\frac{1}{2}$ "	"	=	20
8 flats	$3\frac{1}{2}$x $\frac{1}{2}$	" @ 5.95 "	2 "	3 "	"	=	110
100 flats	$2\frac{1}{2}$x $\frac{3}{8}$	" @ 3.19 "	2 "	$1\frac{1}{2}$ "	"	=	680
8 plates	$8\frac{1}{2}$x $\frac{5}{8}$	" @18.06 "	1 "	6 "	"	=	220
8 plates	$8\frac{1}{2}$x $\frac{5}{8}$	" @18.06 "	2 "		"	=	290
4 plates	$8\frac{1}{2}$x $\frac{9}{16}$	" @16.26 "	2 "		"	=	130
8 plates	39 x $\frac{5}{8}$	" @82.88 "	5 "		"	=	3,320
8 plates	33 x $\frac{9}{16}$	" @63.12 "	5 "	6 "	"	=	2,720
Bottom Chords.							
8 plates	18 x $\frac{3}{8}$	" @22.96 "	24 "	$7\frac{1}{2}$ "	"	=	4,530
16 angles	$3\frac{1}{2}$x$3\frac{1}{2}$ x $\frac{3}{8}$	" @ 8.50 "	23 "	$7\frac{1}{2}$ "	"	=	3,220
4 plates	18 x $\frac{5}{8}$	" @38.26 "	53 "	3 "	"	=	8,120
8 angles	6 x$3\frac{1}{2}$ x $\frac{5}{8}$	" @18.90 "	55 "	3 "	"	=	8,350
8 plates	35 x $\frac{3}{4}$	" @89.25 "	3 "	$4\frac{1}{2}$ "	"	=	2,410
8 plates	38 x $\frac{1}{2}$	" @64.60 "	5 "	$10\frac{1}{2}$ "	"	=	3,040
4 plates	38 x $\frac{1}{2}$	" @64.60 "	4 "	9 "	"	=	1,230
8 plates	18 x $\frac{1}{4}$	" @15.30 "	1 "	9 "	"	=	220
8 flats	$3\frac{1}{2}$x $\frac{1}{2}$	" @ 5.95 "	1 "	9 "	"	=	80
20 plates	21 x $\frac{3}{8}$	" @26.78 "	1 "	9 "	"	=	940
2 plates	21 x $\frac{3}{8}$	" @26.78 "	1 "		"	=	50
308 flats	$2\frac{1}{2}$x $\frac{3}{8}$	" @ 3.19 "	2 "	$1\frac{1}{2}$ "	"	=	2,090
Vertical Posts.							
4 plates	$11\frac{1}{2}$x $\frac{3}{8}$	" @14.68 "	12 "		"	=	700
8 angles	$3\frac{1}{2}$x 3 x $\frac{3}{8}$	" @ 7.80 "	12 "			=	750
8 angles	$3\frac{1}{2}$x 3 x $\frac{3}{8}$	" @ 7.80 "	11 "	9 "	"	=	730
6 plates	$11\frac{1}{2}$x $\frac{3}{8}$	" @14.68 "	12 "	3 "	"	=	1,080
24 angles	$3\frac{1}{2}$x 3 x $\frac{3}{8}$	" @ 7.80 "	12 "	3 "	"	=	2,300
4 plates	$8\frac{1}{2}$x $\frac{3}{4}$	" @21.68 "		6 "	"	=	40
Diagonals.							
4 plates	$11\frac{1}{2}$x $\frac{1}{2}$	" @19.55 "	18 "	$10\frac{1}{2}$ "	"	=	1,470
8 angles	6 x 4 x $\frac{3}{4}$	" @23.60 "	19 "	3 "	"	=	3,630
8 angles	6 x 4 x $\frac{3}{4}$	" @23.60 "	18 "	$10\frac{1}{2}$ "	"	=	3,560
4 plates	$11\frac{1}{2}$x $\frac{3}{8}$	" @14.68 "	16 "	9 "	"	=	980
16 angles	6 x$3\frac{1}{2}$ x $\frac{9}{16}$	" @17.10 "	16 "	9 "	"	=	4,590
8 plates	$11\frac{1}{2}$x $\frac{3}{8}$	" @14.68 "	16 "	4 "	"	=	1,920
16 angles	6 x$3\frac{1}{2}$ x $\frac{3}{8}$	" @11.70 "	16 "	4 "	"	=	3,060
16 angles	6 x$3\frac{1}{2}$ x $\frac{3}{8}$	" @11.70 "	16 "	4 "	"	=	3,060
							109,430
Rivet-heads (4%)						=	4,370
						lbs.	113,800

ONE END BRACE-FRAME.

Top Strut.					
2 angles	3½x3 x$\frac{5}{16}$-in.	@ 6.60 lbs.	9 ft. long	=	120
Bottom Strut.					
4 angles	3 x3 x$\frac{5}{16}$	" @ 6.10 "	8 " 6 ins. "	=	210
2 plates	18 x ⅜	" @22.95 "	1 " 3 " "	=	60
6 flats	2½x ⅜	" @ 3.19 "	1 " 9 " "	=	30
Diagonals.					
2 angles	6 x3½x ⅜	" @11.70 "	12 " "	=	280
Connections.					
4 angles	3½x3½x ⅜	" @ 8.50 "	1 " 1 " "	=	40
8 angles	3½x3½x$\frac{5}{16}$	" @ 7.10 "	1 " 4½ " "	=	80
4 angles	3½x3 x$\frac{5}{16}$	" @ 6.60 "	1 " 4½ " "	=	40
Gussets.					
2 plates	18 x ⅜	" @22.95 "	1 " 7½ " "	=	80
2 plates	17 x ⅜	" @21.68 "	1 " 9 " "	=	80
					1,020
Rivet-heads (2%)				=	20
				lbs.	1,040

ONE INTERMEDIATE BRACE-FRAME.

Struts.					
4 angles	3½x3 x$\frac{5}{16}$-in.	@ 6.60 lbs.	9 ft. long	=	240
Diagonals.					
2 angles	3 x3 x$\frac{5}{16}$	" @ 6.10 "	12 " 7½ ins. "	=	150
Connections.					
8 angles	3½x3½x$\frac{5}{16}$	" @ 7.10 "	10½ " "	=	50
Gussets.					
4 plates	12 x ⅜	" @15.30 "	1 " 1½ " "	=	70
					510
Rivet-heads (2%)				=	10
				lbs.	520

TOP LATERALS.

1st Diagonals.					
4 angles	6 x3½x ⅜-in.	@11.70 lbs.	12 ft. 9 ins. long	=	600
Connections.					
8 angles	3½x3½x ⅜	" @ 8.50 "	9 " "	=	50
2d Diagonals.					
4 angles	5 x3 x ⅜	" @ 9.80 "	12 " 9 " "	=	500
Connections.					
8 angles	3 x3 x ⅜	" @ 7.20 "	9 " "	=	40
3d Diagonals.					
4 angles	3 x3 x ⅜	" @ 7.20 "	12 " 9 "	=	370
Connections.					
8 angles	3 x3 x ⅜	" @ 7.20 "	7½ " "	=	30
			Forward		1,590

			Brought forward		1,590
4th Diagonals.					
4 angles	3 x3 x$\frac{5}{16}$-in.	@ 6.10 lbs	12 ft. 9 ins. long	=	310
Gussets.					
4 plates	15 x $\frac{3}{8}$ "	@19.13 "	2 " 6 " "	=	190
4 plates	14 x $\frac{3}{8}$ "	@17.85 "	4 " 9 " "	=	340
4 plates	13 x $\frac{3}{8}$ "	@16.58 "	3 " 11 " "	=	260
4 plates	12 x $\frac{3}{8}$ "	@15.30 "	4 " $1\frac{1}{2}$ " "	=	250
2 plates	12 x $\frac{3}{8}$ "	@15.30 "	4 " "	=	120
Struts.					
8 angles	$3\frac{1}{2}$x3 x$\frac{5}{16}$ "	@ 6.60 "	8 " 9 " "	=	460
					3,520
Rivet-heads (2%)				=	70
				lbs.	3,590

BOTTOM LATERALS

Diagonals.					
16 angles	3 x3 x$\frac{5}{16}$-in.	@16.10 lbs.	12 ft. 9 ins. long	=	1,240
Gussets.					
4 plates	15 x $\frac{3}{8}$ "	@19.13 "	2 " 3 " "	=	170
14 plates	12 x $\frac{3}{8}$ "	@15.30 "	4 " "	=	850
Struts.					
8 angles	$3\frac{1}{2}$x3 x$\frac{5}{16}$ "	@ 6.60 "	8 " 9 " "	=	460
					2,720
Rivet-heads (2%)				=	60
				lbs.	2,780

PIER-MEMBERS.

Shoes.					
4 plates	39 x$1\frac{3}{8}$-in.	@182.32 lbs.	2 ft. 9 ins. long	=	2,000
Roller-Beds.					
2 plates	33 x$1\frac{3}{8}$ "	@154.27 "	2 " 9 " "	=	850
Rollers.					
10 rounds	5-in. diam.	@ 66.76 "	2 " 3 " "	=	1,500
Spacing-Bars.					
4 flats	$4\frac{1}{2}$x $\frac{1}{2}$-in.	@ 7.65 "	2 " 9 " "	=	80
Anchors.					
8 rounds	$1\frac{1}{4}$-in. diam.	@ 4.17 "	1 " 9 " "	=	60
Guards.					
4 angles	$4\frac{1}{2}$x3 x $\frac{3}{8}$-in.	@ 9.10 "	2 " "	=	70
Cast Bed-Plates,	2x33 ins.x33 ins.x6.25 ins.	= 13.612			
(less voids)	2x25x5.2 ins.x5.2 ins.x5 ins.	= 6,750			
		6,872 cu. ins.		=	1,780
				lbs.	6,340

SUMMARY.

2 Trusses	= 113,800
2 End Brace-Frames @ 1,040	= 2,080
3 Intermediate Brace-Frames @ 520	= 1,560
Top Laterals	= 3,590
Bottom Laterals	= 2,780
Pier-Members	= 6,340
	130,150 lbs.

Total weight of steel, not including pier-members, = 123,810 lbs. = 1,238 lbs. per lin. ft. The assumed weight per foot was 1,200 lbs.

The above estimate should not vary more than 2% from the actual shipping weight of bridge; but, usually, an estimate is required before any detail drawings can be made; in which case, the customary method of procedure is as follows: The weight of a bar of iron one inch square and three feet long is 10 lbs.; and of steel, 2% more. Then, the weight of the trusses will be equal to the gross areas of all the members, multiplied by their lengths, multiplied by $\frac{10}{3}$, plus from 25% to 50% for details. This percentage varies with the form of truss and style of details, and can be determined approximately from similar structures of known weight. Stringers, floorbeams, laterals, etc. are usually estimated in detail, assuming certain sizes for connection-angles and gusset-plates.

WEIGHT OF TWO TRUSSES.

Top Chords.	$AD = 52.50$ sq. ins.	× 37.5 ft.	× $\frac{10}{3}$	× 4	=	26,200
"	$DF = 58.93$ "	× 25	× "	× 2	=	9,820
Bottom Chords	$ac = 23.42$ "	× 25	× "	× 4	=	7,810
"	$ce = 44.70$ "	× 25	× "	× 4	=	14,900
Diagonals.	$aB = 33.51$ "	× 17.67	× "	× 4	=	7,900
"	$Bc = 24.43$ "	× 17.67	× "	× 4	=	5,760
"	$cD = 17.99$ "	× 17.67	× "	× 4	=	4,250
"	$De = 17.99$ "	× 17.67	× "	× 4	=	4,250
Vertical Posts.	$= 13.51$ "	× 12.5	× "	× 10	=	5,630
						86,520
Details, 31.5% of 86,520					=	27,280
Previous estimate in detail					=	113,800 lbs.

The percentage for details derived above may be used in estimating the weight of trusses of similar design to the one just considered.

CHAPTER IV.

THE DESIGN OF A 150-FT. THROUGH PRATT TRUSS.

This form of truss, when constructed with parallel chords, is most suitable for spans of from 125 ft. to 175 ft. For longer spans, a truss with a curved top chord, similar to the one treated of in Chapter V, will usually be found more economical.

The general dimensions of the structure considered in this chapter are as follows:

Length, 150 ft. c. to. c of end-bearings, = 6 panels of 25 ft.
Depth, 30 ft. c. to c. of chords.
Length of diagonals = $\sqrt{25^2 + 30^2}$ = 39.05 ft.
Width, 16 ft. clear; 17 ft. 9 ins. c. to c. of trusses.
Stringers, 2 lines, 8 ft. c. to c.

The economical depth will be found to be about one-sixth of the span. In the present example, however, the depth must be great enough to give the clear height of 22 ft. 6 ins. above base of rail required by the specification, and to permit of an efficient portal strut. It is usually necessary to make some rough calculations in order to determine the depth of stringers and floorbeams, and thus find the distance from the center of bottom chord to the base of rail.

The width c. to c. of trusses, is determined by the width of the top chord and end-posts.

The weight of floor, in accordance with specification, is taken at 600 lbs. per lin. ft.; and the weight of the steel is determined by the formula $10l + 100$, thus: $(10 \times 150) + 100 = 1{,}600$ lbs. per lin. ft. Then,

$$\begin{aligned} \text{Dead-load (floor)} &= 600 \\ \text{(steel)} &= 1{,}600 \\ \hline \text{(total)} &= 2{,}200 \text{ lbs. per lin. ft.} \end{aligned}$$

Live-load as per specification.

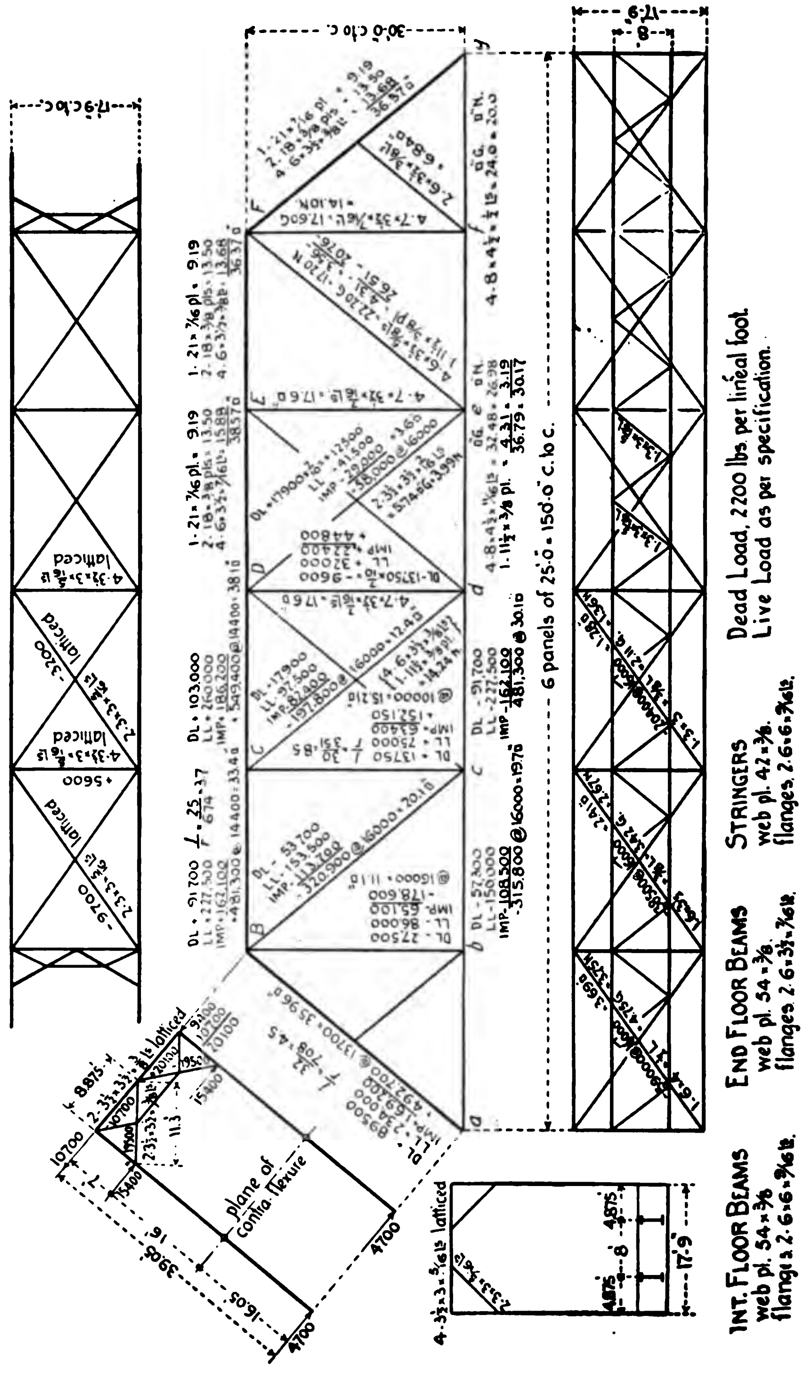


FIG. 9.—150-ft. Through Pratt Truss.

DEAD-LOAD STRESSES.

The dead-load stresses will be computed analytically. The panel dead-load for one truss $= \frac{2,200}{2} \times 25 = 27,500$ lbs., which is assumed to be concentrated at the lower intermediate panel-points. The concentrations at the end panel-points *a* and *g* are equal to one-half panel-load, = 13,750 lbs.; but these affect only the pier-members. The total dead-load reaction is equal to 3 panel-loads, = 27,500 × 3 = 82,500 lbs.; and the dead-load reaction used in computing the stresses is equal to 2½ panel loads, = 27,500 × 2½ = 68,750 lbs. The shears and moments are computed in the same manner as those for the Warren girder, Chap. III. The stresses in members *a B*, *B c*, and *C d* are equal respectively to the shears in panels *a b*, *b c*, and *c d*, multiplied by the length of diagonal, and divided by the depth of truss. *B b* is simply a hanger, and the stress in it is equal to the panel-concentration, at *b*. The stress in *C c* is equal to the shear in panel *c d*. There is no dead-load stress in *D d*, except that due to its own weight with the weight of top chord and laterals, which is small and may be neglected. The stress in *a b c* is equal to the moment at *B* divided by the depth of truss. The stress in *B C* and *c d* is equal to the moment at *c* or *C*, divided by the depth of truss. And the stress in *C D E* is equal to the moment at *d*, divided by the depth of truss. There is no definite stress in the diagonal member between panel-point *b* and the center of the end post *a B*; it is a stiffening member only. The dead-load stresses will now be tabulated.

TABLE OF DEAD-LOAD STRESSES.

Shear in panel	$a\,b =$	$68,750 - 0$	$= 68,750$
" "	$b\,c =$	$68,750 - 27,500$	$= 41,250$
" "	$c\,d =$	$68,750 - (27,500 \times 2)$	$= 13,750$
Moment at panel-point	$B =$	$68,750 \times 25$	$= 1,718,750$
" "	$c\&C =$	$(68,750 \times 50) - (27,500 \times 25)$	$= 2,750,000$
" "	$d =$	$(68,750 \times 75) - [27,500 \times (25+50)]$	$= 3,093,000$
Stress in member	$a\,B =$	$68,750 \times \frac{39.05}{30}$	$= +\ 89,500$
" "	$B\,c =$	$41,250 \times$ "	$= -\ 53,700$
" "	$C\,d =$	$13,750 \times$ "	$= -\ 17,900$
" "	$C\,c =$	$13,750 \times 1$	$= +\ 13,750$
" "	$B\,b =$	$27,500 \times 1$	$= -\ 27,500$

Stress in member	$a\,b\,c = 1{,}718{,}750 \times \frac{1}{30}$		$= - 57{,}300$
"	"	$B\,C = 2{,}750{,}000 \times$ "	$= + 91{,}700$
"	"	$c\,d = 2{,}750{,}000 \times$ "	$= - 91{,}700$
"	"	$C\,D\,E = 3{,}093{,}000 \times$ "	$= + 103{,}200$

Live-Load Stresses.

The live-load shears and moments will be obtained from the moment-diagram, as in the previous examples, first constructing a diagram of the truss on tracing cloth, to the same scale as the moment-diagram. The rules, given in Chapter III, for finding the positions of the live-load for maximum shears and moments in the deck Warren girder apply to the present example.

For the maximum shear in panel $a\,b$, as well as for the maximum moment at B, the diagram of truss is placed on the moment-diagram with wheel 4 at b, as shown in Fig. 10. In this position, the load in the panel to the left of b = 57,500 lbs.; and the total load on span = 389,750 lbs., which values are obtained from the summation of loads on moment-diagram. Then 57,500 lbs. is less than 389,750 ÷ 6 = 64,960 lbs. In other words, the load in the panel to the left of b is less than the total load on span, divided by the number of panels in span. With wheel 5 at b, the load in the panel to the left of this point would be 80,500 lbs.; and the total load on span would be 401,000 lbs. Since 80,500 is greater than 401,000 ÷ 6 = 66,830, the maximum shear in panel $a\,b$ does not occur with wheel 5 at b, but with wheel 4 at this point, as shown. Verticals are dropped from a and g, intersecting the equilibrium-polygon in a_1 and g_1, through which points the closing line is drawn. Then a line drawn through the point O in force-polygon, parallel with the closing line $a_1\,g_1$, and intersecting the vertical load-line, determines the reaction at a for this position of the load, which reaction is equal to the part of the load-line above this intersection. This reaction is greater than the shear in panel $a\,b$; for a portion of the load in this panel is carried to the abutment by the end stringer, and must be deducted from the reaction to obtain the shear. The reaction of the stringer at a is determined by drawing a line through the point O in force-polygon, parallel with a closing line $a_1\,b_1$, and intersecting the load-line. It is represented by the length of the load-line above this last intersection. Then the

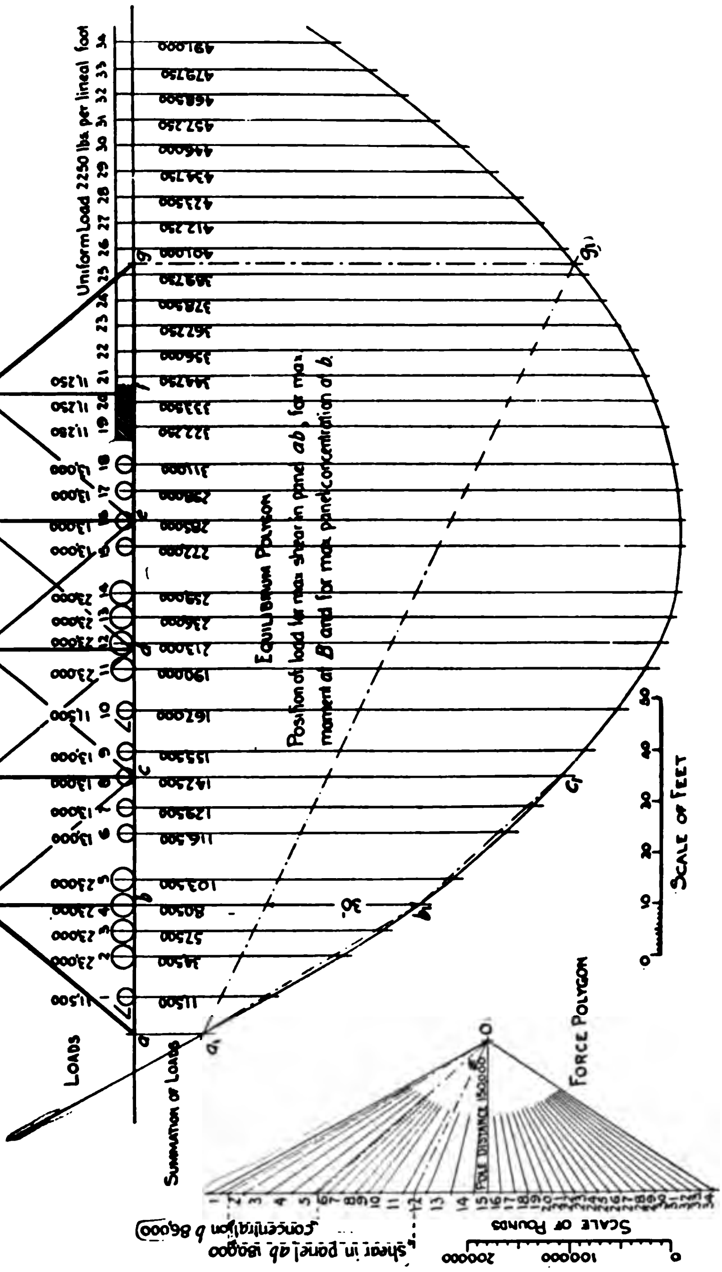


Fig. 10.—150-ft. Through Pratt Truss.

shear in panel $a\ b$ is equal to the total reaction, less the reaction of the end stringer, = 180,000 lbs., as shown. This position of the load also gives the maximum panel concentration at b, which is determined by lines drawn through the point O in force-polygon, parallel with closing lines $a_1\ b_1$ and $b_1\ c_1$, and intersecting the load-line. Then the concentration at b is equal to the length of load-line between these two intersections, = 86,000 lbs., as shown The ordinate at b, between the main closing line and the equilibrium-polygon scales 30 ft. Then, 30 ft. × 150,000 lbs. = 4,500,000 ft.-lbs., which is the maximum moment at this point.

For the maximum shear in panel $b\ c$, the diagram of the truss is placed on the moment-diagram with wheel 3 at c, as shown in Fig. 11. In this position the load in the panel to the left of c = 34,500 lbs.; and the total load on span = 322,250 lbs. Then 34,500 lbs. is less than 322,250 ÷ 6 = 53,700 lbs. With wheel 4 at c, the load in the panel to the left of this point would be 57,500 lbs.; and the total load on span would be 333,500 lbs. Then, since 57,500 is greater than 333,500 ÷ 6 = 55,600, the maximum shear in panel $b\ c$ does not occur with wheel 4 at c but with wheel 3 at this point, as shown. Verticals are drawn through a and g, intersecting the equilibrium-polygon in a_1 and g_1, through which points the closing line is drawn; and verticals are also drawn through b and c, intersecting the equilibrium-polygon in b_1 and c_1, and a closing line drawn through these latter points. Then, lines drawn through the point O in force-polygon, parallel with $a_1\ g_1$, and $b_1\ c_1$, and intersecting the vertical load-line, determine the shear in panel $b\ c$, which is equal to the portion of the load-line between these two intersections, = 118,000 lbs., as shown.

The maximum shear in panel $c\ d$, which occurs with wheel 3 at d, = 75,000 lbs.

The maximum shear in panel $d\ e$, with reference to the left-hand support, and which occurs with wheel 2 at e, = 32,000 lbs.

The maximum moment at c is found with wheel 8 at this point, as shown in Fig. 10. The total load to the left of c divided by the number of panels between this point and the left-hand abutment, is just less than the total load on span divided by the number of panels in span, thus: 129,500 ÷ 2 = 64,750 lbs. is less than 389,750 ÷ 6 = 64,960 lbs. Then, dropping

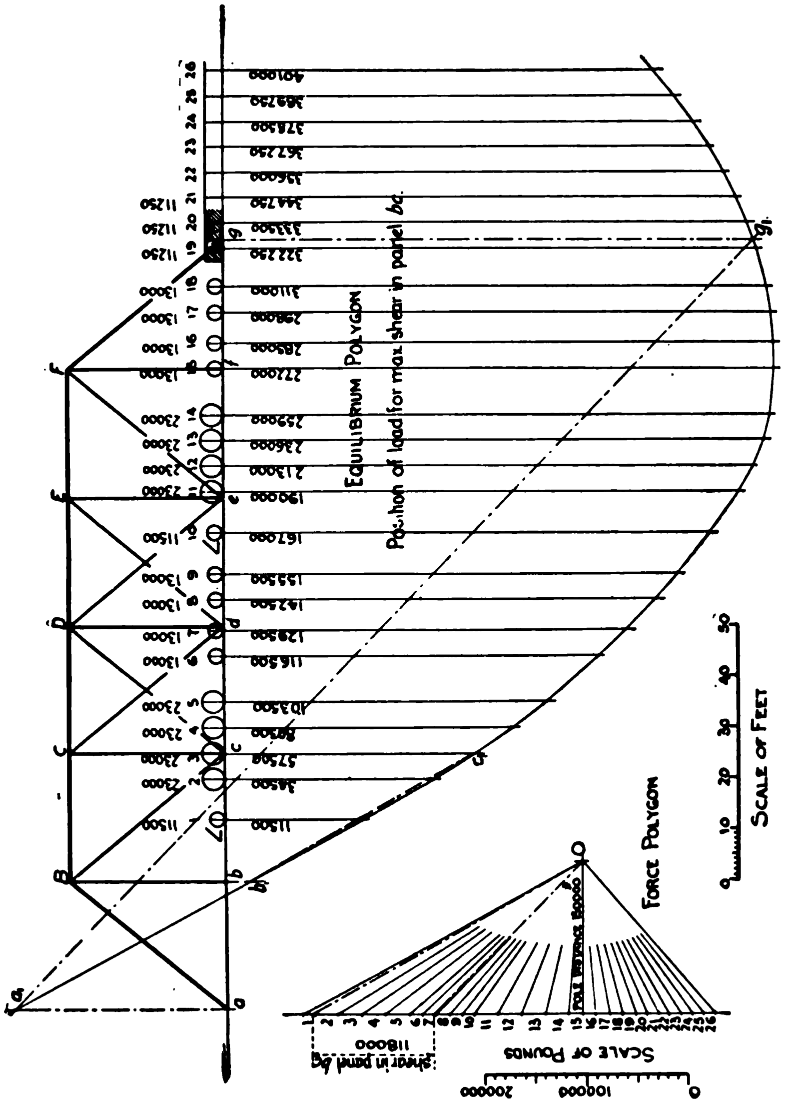


FIG. 11.—150-ft. Through Pratt Truss.

verticals through a and g to intersect the equilibrium-polygon, and drawing the closing line, the ordinate at c between the closing line and the equilibrium-polygon is found to be 45.5 ft.; and the maximum moment at this point = 45.5 ft. × 150,000 lbs. = 6,825,000 ft.-lbs.

The maximum moment at d is found with wheel 12 at this point. In this position, the total load to the left of d = 190,000 lbs., and the total load on span = 389,750 lbs. Then, 190,000 ÷ 3 = 63,300 lbs. is less than 389,750 ÷ 6 = 64,960 lbs. The ordinate between the equilibrium-polygon and the closing line at d scales 52 ft.; and the moment at this point = 52 ft. × 150,000 lbs. = 7,800,000 ft.-lbs.

TABLE OF LIVE-LOAD STRESSES.

Stress in member	$a B$ =	$180{,}000 \times \frac{39.05}{30}$	= +234,000
" "	$B c$ =	$118{,}000 \times$ "	= −153,500
" "	$C c$ =	$75{,}000 \times 1$	= + 75,000
" "	$C d$ =	$75{,}000 \times \frac{39.05}{30}$	= − 97,500
" "	$D d$ =	$32{,}000 \times 1$	= + 32,000
" "	$D e$ =	$32{,}000 \times \frac{39.05}{30}$	= − 41,500
" "	$B b$ =	$86{,}000 \times 1$	= − 86,000
" "	$a b c$ =	$4{,}500{,}000 \times \frac{1}{30}$	= −150,000
" "	$B C$ =	$6{,}825{,}000 \times$ "	= +227,500
" "	$c d$ =	$6{,}825{,}000 \times$ "	= −227,500
" "	$C D E$ =	$7{,}800{,}000 \times$ "	= +260,000

The dead- and live-load stresses are summarized on the stress-diagram, Fig. 9, and the impact stresses added according to specification.

There is no dead-load stress in the center post $D d$, or in the counter-ties $D e$ and $D c$; neither will there be any stresses in these members from the live-load when covering the span; but, with the maximum positive shear in panel $d e$ (with reference to reaction a), which occurs with wheel 2 at e when the train is advancing towards a, post $D d$ will be in compression and the

counter-tie $D\,e$ in tension. With this latter condition of loading, the main tie $d\,E$ will be slackened up, and one-half of the panel-load at d, which goes to the support at g, must go up the post $D\,d$ and down the counter-tie $D\,e$, thus relieving the live-load compression in the one and the live-load tension in the other; but in accordance with clause 10 of specification, only 70% of the effect of the dead-load is considered to counteract the live-load stresses in these members. Thus the resulting computed stresses in $D\,d$ and $D\,e$ are due to live-load only; and, therefore, the impact stresses are equal to the net live-load stresses. The stresses in these members are summarized as follows:

POST $D\,d$, Dead-load $= -13{,}750 \times \frac{7}{10} = -\ 9{,}600$

Live-load $= +32{,}000$

Impact $=$ 32,000–9,600 $= +22{,}400$

Total $= +44{,}800$

COUNTER-TIE $D\,e$, Dead-load $= +\ 17{,}900 \times \frac{7}{10} = +\ 12{,}500$

Live-load $= -\ 41{,}500$

Impact $= -41{,}500 + 12{,}500 = -\ 29{,}000$

Total $= -\ 58{,}000$

The struts from panel-points b and f to the center of end-posts are stiffening members only, and usually called " collision struts."

LATERALS.

The top laterals are to be designed for a fixed horizontal force of 150 lbs. per lin. ft.; and the bottom laterals for a fixed horizontal force of 150 lbs. per lin. ft., plus a moving horizontal force of 400 lbs. per lin. ft. The top-lateral system consists of a horizontal truss of four panels, supported laterally at B and F by the portal struts. The bottom-lateral system consists of a horizontal truss of six panels. Panel dead-load for top and bottom laterals = 150 lbs. × 25 ft. = 3,750 lbs. Panel live-load for bottom laterals only = 400 lbs. × 25 ft. = 10,000 lbs. Length of diagonals $= \sqrt{17.75^2 + 25^2} = 30.6$ ft.

Top Lateral Stresses.

Shear in Panels	Stress in Diagonals.
$BC = 3{,}750 \times 1\frac{1}{2} = 5{,}625$	1st $= 5{,}625 \times \dfrac{30.6}{17.75} = -9{,}700$
$CD = 3{,}750 \times \frac{1}{2} = 1{,}875$	2d $= 1{,}875 \times$ " $= -3{,}200$

Bottom Lateral Stresses.

Shear in Panels.

$$ab = (3{,}750 \times 2\tfrac{1}{2}) + (10{,}000 \times 15/6) = 34{,}400$$
$$bc = (3{,}750 \times 1\tfrac{1}{2}) + (10{,}000 \times 10/6) = 22{,}300$$
$$cd = (3{,}750 \times \tfrac{1}{2}) + (10{,}000 \times 6/6) = 11{,}900$$

Stress in Diagonals.

$$\text{1st} = 34{,}400 \times \frac{30.6}{17.75} = -59{,}000$$
$$\text{2d} = 22{,}300 \times \text{ " } = -38{,}500$$
$$\text{3d} = 11{,}900 \times \text{ " } = -20{,}400$$

Portal Strut.

There are $2\frac{1}{2}$ panel loads of wind force applied at the top of portal strut $= 3{,}750 \times 2\frac{1}{2} = 9{,}400$ lbs. This force is assumed to be resisted equally at the foot of each post. It is also assumed that the posts are fixed at the bottom, and that the plane of contraflexure is half-way between the foot of posts and the lower extremities of portal strut. Then, for the purpose of figuring the portal stresses, the ends of the posts may be considered to lie in this plane, as shown in Fig. 9. The horizontal reaction at the foot of each post $= 9{,}400 \times \frac{1}{2} = 4{,}700$ lbs.; and the bending moments at the knee-connections due to these forces $= 4{,}700 \times 16$ ft. $= 75{,}200$ ft.-lbs. These moments are resisted by forces at the top of posts acting with lever-arms of 7 ft., which forces $= 75{,}200 \div 7 = 10{,}700$ lbs. The force of 10,700 on the leeward side of portal induces a tensile stress of the same amount in this side of the top strut; and, on the windward side, the force of 10,700 combined with the applied-force of 9,400 induces a compressive stress $= 10{,}700 + 9{,}400 = +\ 20{,}100$ lbs.

The horizontal force at the lower end of each knee-brace is equal to the induced force at top of post, plus the horizontal

reaction at its foot, = 10,700 + 4,700 = 15,400 lbs.; and the stress in knee-brace is equal to the horizontal force at its foot, multiplied by its length and divided by one-half the width of portal. $= 15{,}400 \times \frac{11.3}{8.9} = 19{,}500$ lbs. This stress will be tension on the windward side of portal and compression on the leeward side.

PROPORTIONING OF TRUSS-MEMBERS.

For the area required in the tension members, the total stress in each is divided by 16,000; and for the area required in the compression members, the total stress in each is divided by 16,000 reduced by formula for fixed ends, as shown in Fig. 9. For the net area in tension members, allowance has been made for two holes of 1 in. diam. in each angle and each plate.

The end-posts must be capable of resisting the bending moments due to wind force, as well as the direct stresses; but, when these are taken together, the maximum fiber-stress may be 20,000 lbs. per sq. in. The total compression in post = 492,700; and the greatest unsupported length, which is the distance from its foot to the lower extremity of portal strut, = 32 ft. Then, assuming

1 cover plate,	21 x $\frac{7}{16}$-in. =	9.19
2 web-plates,	18 x $\frac{3}{8}$-in. =	13.50
4 angles,	6 x $3\frac{1}{2}$ x $\frac{3}{8}$-in. =	13.68
		36.37 sq. ins.,

the radius of gyration about the neutral axis perpendicular to cover-plate is found to be 7.08 ins.; and the distance from this axis to the outer fibers = $10\frac{1}{2}$ ins. $\frac{l}{r} = \frac{32}{7.08} = 4.5$, which, by Table I, corresponds to a permissible unit stress of 13,700 lbs. per sq. in. for fixed ends.

Area required for direct compression alone = 492,700 ÷ 13,700 = 35.96 sq. ins.

The bending moment due to wind force, as already determined = 75,200 ft.-lbs. = 902,400 in.-lbs. Then, for the combination of direct and bending stresses, the area required is as follows:

Area required for direct compression $= 492{,}000 \div 20{,}000 = 24.63$
Area required for bending moment (Chap. III),

$$= \frac{Mn}{r^2 f} = \frac{902{,}400 \times 10.5}{7.08^2 \times 20{,}000} \qquad = 9.45$$

sq. ins. 34.08

Since this is less than the area required for direct stresses alone, the assumed section of 36.37 sq. ins. will be used.

STRINGERS.

The span of stringers = 25 ft. The dead-load on one stringer consists of one-half the assumed weight of floor plus the weight of stringer, $= 300 + 150 = 450$ lbs. per lin. ft. For the maximum live-load reaction, or end shear, wheel 2 is placed over one support with wheels 3, 4, 5 and 6 on the stringer. Moments of these loads are then taken about the opposite support and divided by the span.

END SHEAR:

Dead-load $= 450 \text{ lbs.} \times 25 \text{ ft.} \times \frac{1}{2} \qquad = 5{,}600$

Live-load $= \dfrac{(13{,}000 \times 1) + 23{,}000\ (10 + 15 + 20 + 25)}{25} = 64{,}900$

Impact $= \dfrac{64{,}900^2}{64{,}900 + 5{,}600} \qquad = 59{,}700$

lbs. 130,200

Area required in web-plate $= 130{,}200 \div 10{,}000 = 13.02$ sq. ins.

A $42 \times \frac{3}{8}$-in. web-plate, = 15.75 sq. ins., will be used.

For the maximum moment the live-load will be four driving-wheels. This maximum moment will not be exactly at the center of span, but under the second driving-wheel, when this wheel and the center of gravity of the four driving-wheels are equidistant from the center of span. The center of gravity of the four driving-wheels is 2.5 ft. to the right of the second driver; therefore the distance from center of span to point of maximum moment $= 2.5 \times \frac{1}{2} = 1.25$ ft. Then, placing the second driver 1.25 ft. to the left of center line, the first driver will be 6.25 ft. from the left support, and the fourth driver will be 3.75 ft. from the right support. The reaction at left support is obtained by taking moments of the four loads about the right support and dividing by the span; and the maximum

moment will be equal to this reaction multiplied by its distance from the second driver, less the weight of first driver multiplied by its distance from the same point.

Left-hand reaction for maximum moment =

$$\frac{23.000\ (3.75+8.75+13.75+18.75)}{25} = 41,400 \text{ lbs.}$$

MOMENT:

Dead-load	=	$\frac{450\times25^2}{8}$	= 35,150
Live-load	=	$(41,400\times11.25)-(23,000\times5)$	= 350,750
Impact	=	$\frac{350,750^2}{350,750+35,150}$	= 318,700
			704,600 ft.-lbs.

The effective depth of stringer, or distance c. to c. of gravity of flanges, will be about 3.25 ft. Flange-stress = 704,600 ÷ 3.25 = 216,600 lbs.

Flange-area required = 216,600 ÷ 16,000 = 13.54 sq. ins.

Then $\frac{1}{8}$ of 42 x $\frac{3}{8}$-in. web-plate = 1.94

2 angles, 6 x 6 x $\frac{9}{16}$-in. = 11.73 (one hole 1 in. diam. in each angle).

13.67 sq. ins. net.

The following formulas will be found convenient for computing the maximum moment for two, three, or four equal loads, equally spaced, as is usually the case with the driving-wheels of typical locomotives.

For two equal loads, $M = \frac{P}{2l}\left(l-\frac{a}{2}\right)^2.$ (1)

For three equal loads, $M = P\left(\frac{2}{3}l-a\right).$ (2)

For four equal loads, $M = P\left(l-2a+\frac{a^2}{4l}\right).$ (3)

in which M = maximum moment,

P = a concentrated load,

l = length of span,

a = distance between loads.

For stringer in present example, $P = 23{,}000$ lbs., $l = 25$ ft., $a = 5$ ft.

Then, from formula (3), $M = 23{,}000\left(25 - 10 + \frac{25}{100}\right) =$ 350,750 ft.-lbs., as before.

Intermediate Floorbeams.

The effective length of floorbeam is assumed to be equal to the distance c. to c. of trusses, = 17.75 ft. The stringer concentrations are 8 ft. apart, and 4.875 ft. from center of trusses. The weight of floorbeam is assumed to be 3,000 lbs., which is a distributed load. The dead-load concentrations from stringers = 450 lbs. × 25 ft. = 11,250 lbs. The live-load concentrations from stringers, which are equal to the maximum panel-concentration at b, and found graphically in Fig. 10 = 86,000 lbs.

END SHEAR:

$$\begin{array}{lll}
\text{Dead-load} & = (3{,}000 \times \tfrac{1}{2}) + 11{,}250 & = 12{,}750 \\
\text{Live-load} & & = 86{,}000 \\
\text{Impact} & = \dfrac{86{,}000^2}{86{,}000 + 12{,}750} & = \underline{74{,}900} \\
 & & \quad 173{,}650 \text{ lbs.}
\end{array}$$

Area required in web-plate = 173,650 ÷ 10,000 = 17.36 sq. ins. A 54 x $\frac{3}{8}$-in. web-plate, = 20.25 sq. ins., will be used.

MOMENT:

$$\begin{array}{lll}
\text{Dead-load} & = \dfrac{3{,}000 \times 17.75}{8} + (11{,}250 \times 4.875) & = 61{,}150 \\
\text{Live-load} & = 86{,}000 \times 4.875 & = 419{,}000 \\
\text{Impact} & = \dfrac{419{,}000^2}{419{,}000 + 61{,}150} & = \underline{365{,}700} \\
 & & \quad 845{,}850 \text{ ft.-lbs.}
\end{array}$$

Effective depth of floorbeam = 4.25 ft. Flange-stress = 845,850 ÷ 4.25 = 199,000 lbs. Flange-area required = 199,000 ÷ 16,000 = 12.44 sq. ins.

$$\begin{array}{ll}
\text{Then } \tfrac{1}{8} \text{ of } 54\text{x}\tfrac{3}{8}\text{-in. web-plate} & = 2.53 \\
\text{2 angles } 6 \times 6 \times \tfrac{9}{16}\text{-in.} & = \underline{10.61} \text{ (2 holes 1 in. diam. in each angle).} \\
 & \quad 13.14 \text{ sq. ins. net.}
\end{array}$$

END FLOORBEAMS.

The effective length and location of stringer-concentrations are the same as for intermediate floorbeams. The weight of floorbeam = 3,000 lbs. Dead-load concentrations from stringers = 450 lbs. × 12.5 ft. = 5,600 lbs. Live load concentrations from stringers, as determined in connection with stringers, = 64,900 lbs.

END SHEAR:

$$\text{Dead-load} = (3{,}000 \times \tfrac{1}{2}) + 5{,}600 = 7{,}100$$

$$\text{Live-load} = 64{,}900$$

$$\text{Impact} = \frac{64{,}900^2}{64{,}900 + 7{,}100} = 58{,}500$$

130,500 lbs.

Area required in web-plate = 130,500 ÷ 10,000 = 13.05 sq. ins. A 54x$\frac{3}{8}$-in. web-plate = 20.25 sq. ins., will be used.

MOMENT:

$$\text{Dead-load} = \frac{3{,}000 \times 17.75}{8} + (5{,}600 \times 4.875) = 34{,}000$$

$$\text{Live-load} = 64{,}900 \times 4.875 = 316{,}000$$

$$\text{Impact} = \frac{316{,}000^2}{316{,}000 + 34{,}000} = 285{,}000$$

635,000 ft.-lbs.

Effective depth of floorbeam = 4.33 ft. Flange-stress = 635,000 ÷ 4.33 = 146,500 lbs. Flange-area required = 146,500 ÷ 16,000 = 9.17 sq. ins.

Then,

$\frac{1}{8}$ of 54x$\frac{3}{8}$-in. web-plate	= 2.53	
2 angles, 6x3$\frac{1}{2}$x$\frac{7}{16}$-in.	= 7.06	(1 hole 1 in diam. in each angle).
	9.59 sq. ins. net.	

The 3$\frac{1}{2}$-in. legs of flange-angles will be connected with the web-plate, as a single line of rivets will be found sufficient for the longitudinal shearing stresses.

Some prominent bridge engineers advocate making the end floorbeams like the intermediate ones; because, owing to their position, they are subject to very great impact stresses when the live-load first comes on the bridge.

Details.

Fig. 12 is a detail drawing of the bridge showing an inside view of the truss, top and bottom laterals, intermediate floorbeam and top strut, end floorbeam, portal strut, stringers, and pier-members. In order to save space, and at the same time to make the details as clear as possible, the center-line diagrams of truss and laterals are drawn to a smaller scale than the details. The center of gravity of end-post and top-chord sections is made to coincide with the center-line diagram; and the distance from the backs of top angles of these members is found to be about 7½ ins.

Rivets ⅞-in. in diameter are used throughout. The value of one rivet in single shear (Table I, Chap. I) = 6,610 lbs.; and since for field-connections the number of rivets in a joint must be 25% in excess of the number of shop rivets required, the value of one field rivet in single shear = 6,610 × ⅘ = 5,280 lbs.

Detail at A.—The stress in end post = 492,700 lbs. Then 492,700 ÷ 5,280 = 94 field rivets, in single shear, required in end connection; whereas the drawing shows 98 rivets. The main cover-plate of post is cut off just far enough from the end to clear the gusset plates, and the section is reinforced at this point by two angles 4x3½x½-in., which also serve to distribute the connecting rivets more symmetrically about the center-of-gravity lines.

The stress in bottom chord = 315,800 lbs. Then 315,800 ÷ 5,280 = 60 field rivets in single shear required, whereas the drawing shows 72 rivets. The gusset plates must be planed on the bottom edges to bear perfectly on the shoe-plate. The vertical diaphragm is required to distribute the end-floorbeam load between the two gusset plates. The maximum load on end-bearing is as follows:

3 panels of dead-load @ 27,500	= 82,500
Live-load (from moment diagram, wheel 2 at *a*)	= 228,000
Impact = $\dfrac{228,000^2}{228,000+82,500}$	= 168,000
	lbs. 478,500

Spherical bearings are provided to prevent uneven loading of the masonry and secondary stresses in truss members which would otherwise occur if the bridge-seat were slightly out of

level, or with the deflection of the bridge. The upper casting is concave, and made of steel. The lower casting is convex, and made of steel or iron. The two are connected by a 2½-in. bolt which passes through their centers, the hole in the upper casting being large enough to permit of a slight movement. Assuming 5-in. rollers, the permissible bearing per lineal inch $=1{,}200\sqrt{5} = 2{,}680$ lbs. Then $478{,}500 \div 2{,}680 = 178$ lin. ins. required. There are 6 rollers of 30 ins. bearing-length each, = 180 lin. ins. provided. The upper and lower roller-plates are planed with ridges at each side, which serve to guide the rollers and prevent lateral movement. The required area of bed-plates $= 478{,}500 \div 400 = 1{,}196$ sq. ins. The size of the roller-plate, in this instance, is governed by the rollers. Its area $= 39 \times 36 = 1{,}400$ sq. ins. The lower casting at fixed end is made deeper than that at the roller-end by the diameter of the rollers plus the thickness of roller-plate. This arrangement permits of both bridge-seats being built at the same elevation. The area of the fixed-end bed-plate $= 36 \times 36 = 1{,}296$ sq. ins.

Detail at B.—At this point, the end-post and top chord should have enough rivets in the web-connections for the total stresses; for, although the ends of these members are faced, it is not considered advisable to trust to their bearing against one another to transmit stresses. The splice-plates on top and bottom flanges are only counted on for lateral stiffness. The number of rivets required in the various members at this point is as follows:

In $a\,B$, $492{,}700 \div 6{,}610 = 75$ shop rivets.
" $B\,C$, $481{,}300 \div 5{,}280 = 92$ field rivets.
" $B\,c$, $320{,}900 \div 5{,}280 = 61$ " "
" $B\,b$, $178{,}600 \div 5{,}280 = 34$ " "

Detail at C.—In the top-chord splice at this point, the rivets are only intended to hold the members in line, the faced ends of the abutting members being relied on to transmit the stresses from one section to the other.

The number of field rivets required in $C\,c = 152{,}150 \div 5{,}280 = 29$
" " " " " " " $C\,d = 197{,}800 \div 5{,}280 = 38$

Detail at D.—

The number of field rivets required in $D\,d = 44{,}800 \div 5{,}280 = 8$
" " " " " " " $c\,D = 58{,}000 \div 5{,}280 = 12$

Detail at c.

The number of field rivets required in $B\,c$ = 320,900 ÷ 5,280 = 61
" " " " " " " $c\,D$ = 58,000 ÷ 5,280 = 12
" " " " " " " $C\,c$ will be determined by floorbeam reaction.

The bottom chord is spliced 1 ft. 10½ ins. to the left of panel-point c. The stress in $b\,c$ = 315,800 lbs., and the value of rivets on left side of splice is as follows:

48 field rivets in single shear	@ 5,280	=	253,400
8 " " " double "	@ 10,560	=	84,500
			337,900 lbs.

The rivets on the right-hand side of splice should be equal in value to the stress in $c\,d$, = 481,300 lbs.; and the value of these rivets is as follows:

66 shop rivets in single-shear	@ 6,610	=	436,000
8 field " " " "	@ 5,280	=	42,000
8 " " " double "	@ 10,560	=	84,500
			562,500 lbs.

Gusset Plates.—As stated in the previous example, the thickness of gusset plates is determined largely by judgment. Plates ⅝-in. thick would appear to be about right for panel-points a, B, and c, and ½-in. plates elsewhere; but, in order to keep the posts of a uniform width, and, at the same time, to avoid the use of fillers as far as possible, ⅝-in. plates are also used at C and D.

Web-plates 5/16-in. thick are used in all vertical members. These plates do not appear on the stress-diagram for the reason that no metal less than ⅜-in. thick is to be used for main members. But 5/16-in. metal may be used for tie-plates, which these are assumed to be.

The bottom-chord member $c\,d\,e$ has holes on the center line of web-plate, about one foot apart, for drainage.

Short tie-plates are provided for the counter-ties and end-sections of bottom chord to reduce vibration.

Stringers.—For the rivet-spacing in flanges, the stringer is supposed to be divided into 6 equal panels, and the shear computed at the center of each. The maximum live-load shear at

any point is obtained with sufficient accuracy by constructing a parabola, as in Fig. 2, making the maximum ordinate equal to the end shear (64,900 lbs.), as previously determined. The longitudinal shear per lineal inch at the flanges is assumed equal to the vertical shear, divided by the distance in inches between the centers of gravity of the top and bottom flanges; and the amount of this shear to be transferred to the flange-angles is proportional to

$$\frac{\text{net area of one flange}}{\text{net area of one flange} + \frac{1}{8}\text{ of web-plate}} = \frac{11.73}{13.67}.$$

In addition to the longitudinal shear on rivets, those in the top flange are required to support the wheel-concentrations, which are assumed to be distributed over a length of 36 ins. Then the total stress per lineal inch to be resisted by the top-flange rivets will be equal to

$$\sqrt{(\text{longitudinal shear per lineal inch on rivets})^2 + (\text{vertical load per lineal inch})^2}.$$

Rivet-Spacing in First Panel —

Shear at center of panel:

Dead-load $= 5{,}600 - (450 \times 2)$	$=$ 4,700
Live-load (from diagram)	$=$ 55,000
Impact $= \dfrac{55{,}000^2}{55{,}000 + 4{,}700}$	$=$ 50,800
	110,500 lbs.

Distance, c. to c. of gravity of flanges = 39 ins.

Longitudinal shear, per lin. in. at flanges $= \dfrac{110{,}500}{39} = 2{,}830$ lbs.

Longitudinal shear, per lin. in. on rivets $= 2{,}830 \times \dfrac{11.73}{13.67} =$ 2,430 lbs.

Bearing value of one ⅞-in. rivet on ⅜-in. web-plate = 7,220 lbs.

Required spacing in bottom flange $= \dfrac{7{,}220}{2{,}430} = 2.9$ ins.

The maximum wheel-concentration is 23,000 lbs., to which is added 23,000 lbs. impact, making 46,000 lbs. to be distributed over a length of 36 ins., = 1,275 lbs. per lin. in. Total stress, per lin. in. to be resisted by top-flange rivets $= \sqrt{2{,}430^2 + 1{,}275^2}$ = 2,740 lbs.

Required spacing in top flange $= \dfrac{7{,}220}{2{,}740} = 2.6$ ins.

Rivet-Spacing in Second Panel.

Shear at center of panel:

Dead-load = 5,600 − (450×6)	= 2,900
Live-load (from diagram)	= 38,000
Impact $= \dfrac{38{,}000^2}{38{,}000 + 2{,}900}$	= 35,100
	76,000 lbs.

Longitudinal shear, per lin. in., at flanges $= \dfrac{76{,}000}{39} = 1{,}950$ lbs.

Longitudinal shear, per lin. in., on rivets $= 1{,}950 \times \dfrac{11.73}{13.67} =$ 1,670 lbs.

Required spacing in bottom flange $= \dfrac{7{,}220}{1{,}670} = 4.3$ ins.

Total stress, per lin. in., to be resisted by top-flange rivets $= \sqrt{1{,}670^2 + 1{,}275^2} = 2{,}100$ lbs.

Required spacing in top flange $= \dfrac{7{,}220}{2{,}100} = 3.4$ ins.

Rivet-Spacing in Third Panel.—

Shear at center of panel:

Dead-load = 5,600 − (450×10)	= 1,100
Live-load (from diagram)	= 23,000
Impact $= \dfrac{23{,}000^2}{23{,}000 + 1{,}100}$	= 22,000
	46,100 lbs.

Longitudinal shear, per lin. in., at flanges $= \dfrac{46{,}100}{39} = 1{,}180$ lbs.

Longitudinal shear, per lin. in., on rivets $= 1{,}180 \times \frac{11.73}{13.67} =$ 1,010 lbs.

Required spacing in bottom flange $= \frac{7{,}220}{1{,}010} = 7.1$ ins.

Total stress, per lin. in., to be resisted by top-flange rivets $= \sqrt{1{,}010^2 + 1{,}275^2} = 1{,}630$ lbs.

Required spacing in top flange $= \frac{7{,}220}{1{,}630} = 4.4$ ins.

The rivets will be spaced as follows:

In bottom flange: 1st panel, 2½ ins. staggered; 2d panel, 4 ins. staggered; 3d panel, 6 ins. staggered. In top flange: 1st panel, 2½ ins. staggered; 2d panel, 3 ins. staggered; 3d panel, 4 ins. staggered.

End Connections.—The total end shear = 130,200 lbs. Then 130,200 ÷ 7,220 = 18 rivets required in bearing on ⅜-in. web-plate. The drawing shows 15 rivets in the connection-angles, and 6 extra rivets in the filler plates which extend beyond the connection-angles, making 21 rivets in all. The number of rivets required to connect the stringers with the floorbeam will be determined by their bearing value on the web-plate of floor-beam. The ends of the stringers are required to be faced to exact length; and, for this reason, the connection-angles are made extra heavy.

It is intended to use 8 x 12-in. ties on the stringers, notched down ½-in., leaving 11½ ins. from top of stringer to base of rail, as shown.

Intermediate Floorbeams.—The total end shear, as already found, = 173,650 lbs.

The vertical distance between centers of gravity of top and bottom flanges = 51 ins.

Longitudinal shear per lin. in. at flanges $= \frac{173{,}650}{51} = 3{,}400$ lbs.

Area of flange-angles = 10.61; and area of angles + ⅛ of web-plate = 13.14 sq. ins.

Longitudinal shear per lin. in. on rivets $= 3{,}400 \times \frac{10.61}{13.14} =$ 2,740 lbs.

Required spacing of rivets in flanges, between ends and stringer-connections $= \frac{7,220}{2,740} = 2.8$ ins.

There is no vertical shear between the stringer-connections, and, therefore, no horizontal shear on the flange-rivets. These rivets are spaced 6 ins., staggered.

The rivets connecting the end-angles with the floorbeam are in bearing on the $\frac{3}{8}$-in. web-plate. The number required = $173,650 \div 7,220 = 24$. The drawing shows 24 rivets in these angles, and 6 extra rivets in the filler plates which extend beyond the angles, making 30 rivets in all.

The rivets connecting the stringers with the floorbeam are also in bearing on the $\frac{3}{8}$-in. web-plate, but they are to be field-driven. The number required $= 24 + 25\% = 30$, as shown.

The rivets connecting the floorbeam with truss are in single shear and field-driven. The number required $= 173,650 \div 5,280 = 33$. In the connection to the vertical truss-member at b, only the rivets above the bottom-chord angles are efficient, of which there are 36.

The ends of floorbeams are to be faced to exact length.

End Floorbeams.—The total end shear = 130,500 lbs.

Vertical distance between centers of gravity of top and bottom flanges = 52 ins.

Longitudinal shear per lin. in. at flanges $= \frac{130,500}{52} =$ 2,510 lbs.

Area of flange angles = 7.06; and area of angles + $\frac{1}{8}$ of web-plate = 9.59 sq. ins.

Longitudinal shear per lin. in. on rivets $= 2,510 \times \frac{7.06}{9.59} =$ 1,850 lbs.

Required spacing of rivets in flanges, between ends and stringer-connections $= \frac{7,220}{1,850} = 3.9$ ins.

The end gusset plates of trusses, with diaphragms and shoe plates, are shop-riveted to the floorbeam for convenience in erection. The rivets in these connections are in single shear

and the number required = 130,500 ÷ 6,610 = 20. The drawing shows 22.

The brackets, shown in connection with the end stringer, are required to carry the outer tie, adjacent to ballast-wall.

Laterals and Portal Struts.—The connections of these members are designed to resist the stresses shown in Fig. 9.

Camber.—The trusses are cambered by adding $\frac{3}{16}$-in. to each top panel.

Special Angles.—The 8x4½-in. angles used in this design are obtained in Scotland.

Estimated Weight.

Two Trusses.

END-POSTS:

				LENGTH.	
Cover-plates.					
4 plates	21 x $\frac{7}{16}$	-in. @	31.34 lbs.	35 ft. 1 in.	= 4,400
Web-plates.					
8 plates	18 x $\frac{3}{8}$	" @	22.96 "	37 " 7 ins.	= 6,920
Flange-angles.					
8 angles	6 x3½ x $\frac{3}{8}$	" @	11.70 "	37 " 7 "	= 3,520
8 angles	6 x3½ x $\frac{3}{8}$	" @	11.70 "	37 " 3 "	= 3,480
Lock-angles.					
8 angles	4 x3½ x ½	" @	11.90 "	4 " 4 "	= 410
Fillers.					
8 flats	4 x $\frac{3}{8}$	" @	5.10 "	2 " 6 "	= 100
Gussets a.					
8 plates	61 x $\frac{5}{8}$	" @	129.64 "	4 " 10 "	= 5,000
Shoe-connections.					
8 angles	6 x4 x $\frac{5}{8}$	" @	20.00 "	3 "	= 480
Diaphragm.					
4 plates	11½x $\frac{3}{8}$	" @	14.68 "	5 " 1 in.	= 300
16 angles	4 x3½ x $\frac{3}{8}$	" @	9.10 "	5 " 1 "	= 740
Gussets B.					
8 plates	45 x $\frac{5}{8}$	" @	95.64 "	5 " 8 ins.	= 4,330
Hip-covers.					
4 plates	21 x $\frac{3}{8}$	" @	26.78 "	1 " 8 "	180
Tie-plates.					
4 plates	21 x $\frac{3}{8}$	" @	26.78 "	3 " 6 "	= 380
8 plates	21 x $\frac{3}{8}$	" @	26.78 "	1 "	= 210
4 plates	21 x $\frac{3}{8}$	" @	26.78 "	1 " 9 "	= 180
Latticing					
104 flats	2½x $\frac{3}{8}$	" @	3.19 "	2 "	= 670
				Forward	31,300

Fillers.				Brought forward				31,300	
8 flats	6 x 5/8	-in. @	12.75 lbs.	1 ft.			=	100	
4 flats	3 x 3/8	" @	3.83 "		9	ins.	=	10	
4 flats	3 x 3/8	" @	3.83 "	1 "	9	"	=	30	
4 flats	3½x 3/8	" @	4.47 "	1 "	9	"	=	30	
4 flats	3½x 3/8	" @	4.47 "	3 "	6	"	=	60	
									31,530
TOP CHORDS:									
Cover-plates.									
4 plates	21 x 7/16	" @	31.34 "	24 "	6	"	=	3,060	
Web-plates									
8 plates	18 x 3/8	" @	22.96 "	24 "	6	"	=	4,500	
Flange-angles.									
8 angles	6 x3½x 3/8	" @	11.70 "	24 "	6	"	=	2,270	
8 angles	6 x3½x 3/8	" @	11.70 "	24 "			=	2,250	
Cover-plates.									
2 plates	21 x 7/16	" @	31.34 "	51 "	9	"	=	3,240	
Web-plates.									
4 plates	18 x 3/8	" @	22.96 "	51 "	9	"	=	4,770	
Flange-angles.									
8 angles	6 x3½x 7/16	" @	13.50 "	51 "	9	"	=	5,610	
Gussets C									
8 plates	40 x 5/8	" @	85.00 "	4 "	3	"	=	2,890	
Gussets D									
4 plates	30 x 5/8	" @	63.75 "	3 "	9	"	=	960	
Splice B.									
4 flats	3½x 3/8	" @	4.47 "	1 "	7	"	=	30	
Splices C and D.									
6 flats	3½x 3/8	" @	4.47 "	1 "	9	"	=	50	
6 plates	18 x 3/8	" @	22.96 "	1 "	3	"	=	240	
Tie-plates.									
4 plates	21 x 3/8	" @	26.78 "	1 "	9	"	=	190	
12 plates	21 x 3/8	" @	26.78 "	1 "			=	320	
Latticing.									
144 flats	2½x 3/8	" @	3.19 "	2 "			=	920	
Fillers.									
6 flats	6 x 7/16	" @	8.93 "	6 "	6	"	=	30	
									31,330
BOTTOM CHORDS:									
Flanges.									
16 angles	8 x4½x ½	" @	20.40 "	47 "	9	"	=	15,600	
8 angles	8 x4½x 11/16	" @	27.60 "	53 "	9	"	=	11,850	
Webs.									
2 plates	11½x 3/8	" @	14.68 "	56 "			=	1,640	
Tie-plates b.									
4 plates	11½x 3/8	" @	14.68 "	4 "			=	230	
Tie-plates a.									
4 plates	11½x 3/8	" @	14.68 "	2 "	4	"	=	140	
48 plates	11½x 3/8	" @	14.68 "	6 "			=	350	
				Forward				29.810	

Splice c.					Brought forward				29,810	
16 flats	4 x ½	-in.@	6.80	lbs.	2 ft.	3	ins.	=	240	
Gussets b.										
8 plates	27 x ½	" @	45.90	"	2 "			=	740	
Gussets c.										
8 plates	51 x ⅝	" @	108.37	"	5 "	4	"	=	4,630	
Gussets d.										
4 plates	41 x ½	" @	69.80	"	4 "	10	"	=	1,350	
									———	36,770
END POST STRUTS:										
8 angles	6 x3½x ⅜	" @	11.70	"	18 "	6	"	=	1,730	
Tie-plates.										
8 plates	11½x ⅜	" @	14.68	"		9	"	=	90	
Latticing										
60 flats	2¼x ⅜	" @	2.87	"	1 "			=	170	
									———	1,990
VERTICALS:										
B b.										
4 plates	11½x$\frac{5}{16}$	" @	12.22	"	27 "			=	1,320	
8 angles	7 x3½x$\frac{7}{16}$	" @	15.00	"	27 "			=	3,240	
8 angles	7 x3½x$\frac{7}{16}$	" @	15.00	"	26 "	9	"	=	3,210	
Fillers.										
4 plates	14½x ½	" @	24.65	"	2 "	6	"	=	250	
C c and D d.										
6 plates	11½x$\frac{5}{16}$	" @	12.22	"	28 "	10½	"	=	2,120	
24 angles	7 x3½x$\frac{7}{16}$	" @	15.00	"	28 "	10½	"	=	10,400	
Fillers.										
4 plates	14½x ⅝	" @	30.81	"		6	"	=	60	
2 plates	14½x ½	" @	24.65	"	1 "	4	"	=	70	
									———	20,670
DIAGONALS:										
B c.										
16 angles	6 x3½x ⅝	" @	18.90	"	36 "	7	"	=	11,100	
4 plates	11½x ⅜	" @	14.68	"	36 "	7	"	=	2,150	
C d.										
16 angles	6 x3½x ⅜	" @	11.70	"	36 "			=	6,750	
4 plates	11½x ⅜	" @	14.68	"	36 "			=	2,120	
D c.										
8 angles	3½x3½x ⅜	" @	8.50	"	17 "	9	"	=	1,210	
8 angles	3½x3½x ⅜	" @	8.50	"	17 "	6	"	=	1,190	
32 angles	3½x3½x ⅜	" @	8.50	"		10½	"	=	240	
48 plates	11½x ⅜	" @	14.68	"		6	"	=	350	
8 plates	25 x ⅜	" @	31.88	"	2 "	6	"	=	630	
									———	25,740
										148,030
Rivet-heads (3%)								=		4,430
										152,460

ONE END FLOORBEAM.

Web-plate.					
1 plate	54x $\frac{3}{8}$	-in. @ 68.85 lbs.	16 ft. 9 ins. long	=	1,150
Flanges.					
4 angles	6x3½x$\frac{7}{16}$	" @ 13.50 "	16 " 9 " "	=	910
End-connections.					
4 angles	6x3½x $\frac{5}{8}$	" @ 18.90 "	4 " 6 " "	=	340
Fillers.					
4 plates	9x$\frac{7}{16}$	" @ 13.40 "	4 "	=	210
Brackets.					
2 plates	42x $\frac{3}{8}$	" @ 53.55 "	1 "	=	110
4 angles	3½x3½x $\frac{3}{8}$	" @ 8.50 "	1 " 2 " "	=	40
4 angles	3x3 x $\frac{3}{8}$	" @ 7.20 "	2 " 9 " "	=	80
Stiffeners.					
4 angles	4x3½x $\frac{3}{8}$	" @ 9.10 "	4 " 6 " "	=	160
Fillers.					
2 plates	8½x$\frac{7}{16}$	" @ 12.64 "	4 "	=	100
					3,100
Rivet-heads (3%)				=	100
					3,200

ONE INTERMEDIATE FLOORBEAM.

Web-plate.					
1 plate	54x $\frac{3}{8}$	-in. @ 68.85 lbs.	16 ft. 9 ins. long	=	1,150
Flanges.					
4 angles	6x6 x$\frac{9}{16}$	" @ 21.90 "	16 " 9 " "	=	1,470
End-connections,					
4 angles	6x6 x $\frac{5}{8}$	" @ 24.20 "	4 " 6 " "	=	630
Fillers.					
4 plates	9x$\frac{9}{16}$	" @ 17.22 "	3 " 6 " "	=	240
					3,490
Rivet-heads (3%)				=	100
					3,590

ONE STRINGER.

Web-plate.					
1 plate	42x $\frac{3}{8}$	-in. @ 53.55 lbs.	25 ft. long	=	1,340
Flanges.					
4 angles	6x6 x$\frac{9}{16}$	" @ 21.90 "	25 "	=	2,190
End-connections.					
4 angles	6x6 x $\frac{5}{8}$	" @ 24.20 "	3 " 6 ins. "	=	240
Fillers.					
4 plates	9x$\frac{9}{16}$	" @ 17,22 "	2 " 6 " "	=	170
Stiffener					
10 angles	3x3 x$\frac{5}{16}$	" @ 6.10 "	3 " 6 " "	=	210
					4,150
Rivet-heads (3%)				=	120
					4,270

One Panel of Stringer Bracing.

4 angles	3x3 x$\frac{5}{16}$-in.	@ 6.10 lbs.	9 ft.	long	=	220
Gussets.						
2 plates	8x$\frac{5}{16}$	" @ 8.50 "	1 "	"	=	20
3 plates	8x$\frac{5}{16}$	" @ 8.50 "	1 " 8 ins.	"	=	40
						280
Rivet-heads (2%)					=	10
						290

Bottom Laterals.

First Diagonals.						
4 angles	6x4 x ½-in.	@16.20 lbs.	27 ft. 6 ins.	long	=	1,780
Second Diagonals.						
4 angles	6x3½x ⅜	" @11.70 "	27 " 3 "	"	=	1,280
Third Diagonals.						
4 angles	3x3 x ⅜	" @ 7.20 "	27 " 6 "	"	=	920
Connections.						
8 angles	3½x3½x ⅜	" @ 8.50 "	1 " 5 "	"	=	100
8 angles	3½x3½x ⅜	" @ 8.50 "	1 " 1 "	"	=	70
Gussets.						
4 plates	17x ⅜	" @21.68 "	3 " 1 "	"	=	270
4 plates	15x ⅜	" @19.14 "	5 " 3 "	"	=	400
4 plates	13x ⅜	@16.58 "	4 " 11 "	"	=	330
2 plates	13x ⅜	" @16.58 "	4 " 7 "	"	=	150
Connections.						
4 angles	3½x3½x ⅜	" @ 8.50 "	2 " 2 "	"	=	80
4 angles	3½x3½x ⅜	" @ 8.50 "	2 " 4 "	"	=	80
4 angles	3½x3½x ⅜	" @ 8.50 "	1 " 8 "	"	=	60
4 angles	3½x3½x ⅜	" @ 8.50 "	2 " 1 "	"	=	70
4 angles	3½x3½x ⅜	" @ 8.50 "	1 " 8 "	"	=	60
4 angles	3½x3½x ⅜	" @ 8.50 "	1 " 9 "	"	=	60
Fillers.						
4 flats	3½x ½	" @ 5.95 "	1 " 6 "	"	=	40
4 flats	3½x ½	" @ 5.95 "	1 " 7½ "	"	=	40
4 flats	3½x ⅝	" @ 7.44 "	7 " 6 "	"	=	10
						5,800
Rivet-heads (2%)					=	120
						5,920

Top Laterals.

8 angles	3x3 x$\frac{5}{16}$-in.	@ 6.10 lbs.	27 ft. 4 ins.	long	=	1,330
16 angles	3x3 x$\frac{5}{16}$	" @ 6.10 "	13 " 6 "	"	=	1,320
Tie-plates.						
24 plates	9x$\frac{5}{16}$	" @ 9.56 "	1 " 6½ "	"	=	350
			Forward			3,000

Latticing.			Brought forward				3,000
280 flats	2½x$\frac{5}{16}$	-in.@ 2.65 lbs.	1 ft.	9	ins. long	=	1,300
Gussets.							
4 plates	18x$\frac{5}{16}$	" @19.13 "		7½	" "	=	50
4 plates	12x ⅜	" @15.30 "	2 "	4	"	=	140
12 plates	11x$\frac{5}{16}$	" @11.68 "	4 "		"	=	560
8 plates	14x$\frac{5}{16}$	" @14.87 "	1 "	7	" "	=	190
							5,240
Rivet-heads (3%)						=	160
							5,400

ONE PORTAL STRUT.

Top Strut.							
2 angles	3½x3½x ⅜	-in.@ 8.50 lbs.	16 ft.		long	=	270
Diagonals.							
4 angles	3½x3½x ⅜	" @ 8.50 "	9 "		"	=	310
4 angles	3x3 x$\frac{5}{16}$	" @ 6.10 "	4 "	1 -in.	"	=	100
Horizontal.							
2 angles	3x3 x$\frac{5}{16}$	" @ 6.10 "	7 "		"	=	90
Gussets.							
4 plates	12x ⅜	" @15.30 "	1 "	9 ins.	"	=	110
2 plates	12x ⅜	" @15.30 "	1 "	6 "	"	=	50
4 plates	22x$\frac{5}{16}$	" @23.36 "		9 "	"	=	70
2 plates	12x ⅜	" @15.30 "	2 "	3 "	"	=	70
Tie-plates.							
12 plates	19x$\frac{5}{16}$	" @20.20 "		9 "	"	=	180
Latticing.							
48 flats	2½x$\frac{5}{16}$	" @ 2.65 "	1 "	9 "	"	=	220
							1,470
Rivet-heads (3%)						=	40
							1,510

ONE INTERMEDIATE TOP STRUT.

Flanges.							
4 angles	3½ x 3 x$\frac{5}{16}$	-in.@ 6.60 lbs.	16 ft.		long	=	420
Tie-plates.							
2 plates	18½ x $\frac{5}{16}$	" @19.66 "	1 "	4½ ins.	"	=	50
2 plates	12 x ⅜	" @15.30 "	2 "	3 "	"	=	70
Connections.							
4 angles	3x3 x$\frac{5}{16}$	" @ 6.10 "	1 "	5½ "	"	=	40
Latticing.							
16 flats	2½x$\frac{5}{16}$	" @ 2.65 "	2 "		"	=	90
Knees.							
4 angles	3x3 x$\frac{5}{16}$	" @ 6.10 "	7 "	3 "	"	=	180
Connections.							
4 angles	8x3½x ⅜	" @14.25 "	1 "		"	=	60
							910
Rivet-heads (3%)						=	30
							940

PIER MEMBERS.

Shoe-plates.								
4 plates	24x4	ins.	@326.40 lbs.	3 ft.		long	=	3,930
Roller-plates.								
2 plates	39x4½	"	@596.70 "	3 "	4 "	"	=	3,980
4 angles	4½x3x⅜	in.	@ 9.10 "	2 "	5½ "	"	=	90
Rollers.								
12 rounds	5-in. diam.		@ 66.76 "	2 "	9 "	"	=	2,200
Spacing-bars.								
4 flats	4½x ½	in.	@ 7.65 "	3 "		"	=	90
Roller bed-plates.								
2 plates	39x2	ins.	@265.20 "	3 "		"	=	1,590
Fixed bed-plates.								
2 castings	(36x36x14 ins.) − 25 (5.2x5.2x7 ins.) = 26,840 cu. ins. @0 26 lb.						=	7,000
Anchors.								
8 rounds	1½-in. diam.		@ 6.00 lbs.	2 ft. long			=	100
								18,980

SUMMARY.

2	Trusses		=	152,460
2	End Floorbeams	@ 3,200	=	6,400
5	Intermediate Floorbeams	@ 3,590	=	17,950
12	Stringers	@ 4,270	=	51,240
6	Panels Stringer-Bracing	@ 290	=	1,740
	Bottom Laterals		=	5,920
	Top Laterals		=	5,400
2	Portal Struts	@ 1,510	=	3,020
3	Intermediate Top Struts	@ 940	=	2,820
	Pier-members		=	18,980
				265,930 lbs.

The total weight of steel, exclusive of the pier-members = 246,950 lbs.; and the weight per lineal foot = 246,950 ÷ 150 = 1,645 lbs., whereas the assumed weight per foot was 1,600 lbs.

To ascertain the percentage of details in trusses, the weight of the main members will be estimated separately, using the c. to c. lengths, and assuming that a bar of 1 sq. in. cross-section and 3 ft. long weighs 10 lbs., as explained at the end of Chapter III.

ESTIMATE OF WEIGHT OF TRUSSES.

a B	= 36.37 sq. ins.	x 39 ft.	x $\frac{10}{3}$	x 4	=	18,900
B C	= 36.37 "	x 25 "	x $\frac{10}{3}$	x 4	=	12,100
C D	= 38.57 "	x 25 "	x $\frac{10}{3}$	x 4	=	12,850
a b c	= 24.00 "	x 25 "	x $\frac{10}{3}$	x 8	=	16,000
c d	= 36.79 "	x 25 "	x $\frac{10}{3}$	x 4	=	12,300
Posts	= 17.60 "	x 30 "	x $\frac{10}{3}$	x 10	=	17,600
Struts	= 6.84 "	x 19.5 "	x $\frac{10}{3}$	x 4	=	1,780
B c	= 26.51 "	x 39 "	x $\frac{10}{3}$	x 4	=	13,750
C d	= 17.99 "	x 39 "	x $\frac{10}{3}$	x 4	=	9,380
D e	= 5.74 "	x 39 "	x $\frac{10}{3}$	x 4	=	2,850
						117,510
Details = 29.7%					=	34,950
Previous estimate in detail					=	152,460 lbs.

CHAPTER V.

THE DESIGN OF A 200-FT. THROUGH PRATT TRUSS WITH CURVED TOP-CHORD.

The truss shown in Fig. 13 is similar to that of Chapter IV, but the curved top-chord makes the stresses more difficult to compute. This form of truss is most suitable for spans of from 175 ft. to 275 ft. Its outline is pleasing to the eye; and the material in it is economically disposed, for the inclined top-chord transmits a considerable portion of the shear (with a very small increase of section), thus materially lightening the web-members. The principal objection to it is the somewhat greater cost of manufacture, due to the beveled joints of the top-chord. The depth at the hips is determined as in the previous example, viz.: by the clear height required, the depth of portal strut and the depth of floor system. For the sake of appearance, as well as to avoid excessively long compression-members, the top-chord should not be curved too much.

The weight of steel for dead-load, as computed by the formula $(10l+100) = (10\times200)+100 = 2{,}100$ lbs. per lin. ft. It may also be approximated from the weight of the 150-ft. span; for the weight of the floor systems will be nearly proportional to the spans, and the weight of the trusses and bracing to the squares of the spans.

For the 150-ft. span,	the weight of floor system	= 77,330
	the weight of trusses and bracing	= 169,620
	total weight, exclusive of pier-members	= 246,950 lbs.

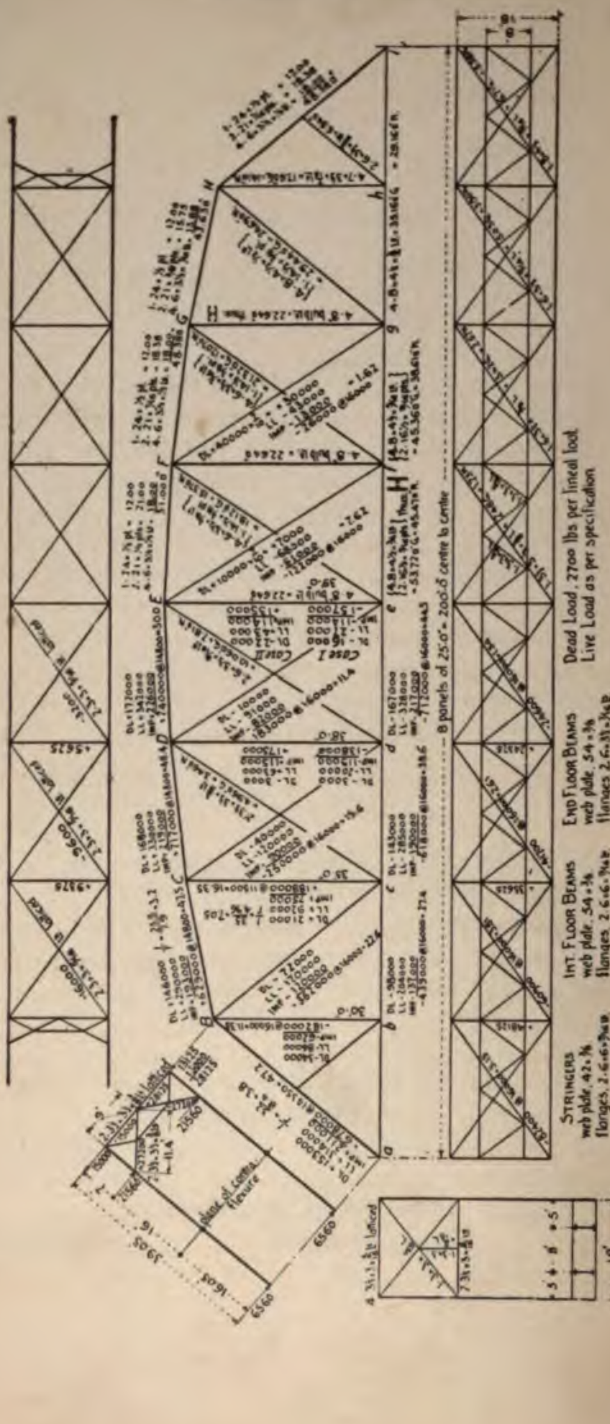


FIG. 13.—200-ft. Through Pratt Truss.

Then, for the 200-ft. span,

the weight of floor system $= 77{,}330 \times \frac{200}{150}$ $= 103{,}100$

the weight of trusses and bracing $= 169{,}620 \times \frac{200^2}{150^2}$ $= 301{,}200$

total weight, exclusive of pier-members $= 404{,}300$ lbs.;

and $404{,}300 \div 200 = 2{,}020$ lbs. per lin. ft. For the stresses, 2,100 lbs. per lin. ft. will be assumed.

The general data are as follows:

Length, 200 ft., c. to c. of end-bearings = 8 panels of 25 ft.

Depth, 30 ft., c. to c. of chords at hips; 39 ft. at center of span (parabolic top-chord).

Width, 16 ft. clear; 18 ft., c. to c. of trusses.

Two lines of stringers, 8 ft. c. to c.

Dead-Load: (floor) 600
(steel) 2,100
(total) 2,700 lbs. per lin. ft.

Live-load as per specification.

DEAD-LOAD STRESSES.

The dead-load stresses will be obtained graphically. The panel load for one truss $= \frac{2{,}700}{2} \times 25 = 33{,}800$ lbs., which is assumed to be concentrated at each lower panel-point.

Fig. 14 consists of a diagram of the truss and two stress-diagrams for dead-load. Letters a to i are used to denote the lower panel-points; B to H the upper panel-points; and K to S the loads and reactions. In stress-diagram, Case I, it is assumed that all the main ties are in action; which is true, either with no live-load on span, or with live-load over all. In stress-diagram, Case II, it is assumed that the counter-ties in panels ef and fg are in action, and the main ties in these panels idle; which is true when the positive live-load shear in these panels, with reference to reaction a, exceeds the negative dead-load shear. In practice, only one diagram, Case II, need be drawn. There is no difficulty in constructing the stress-dia-

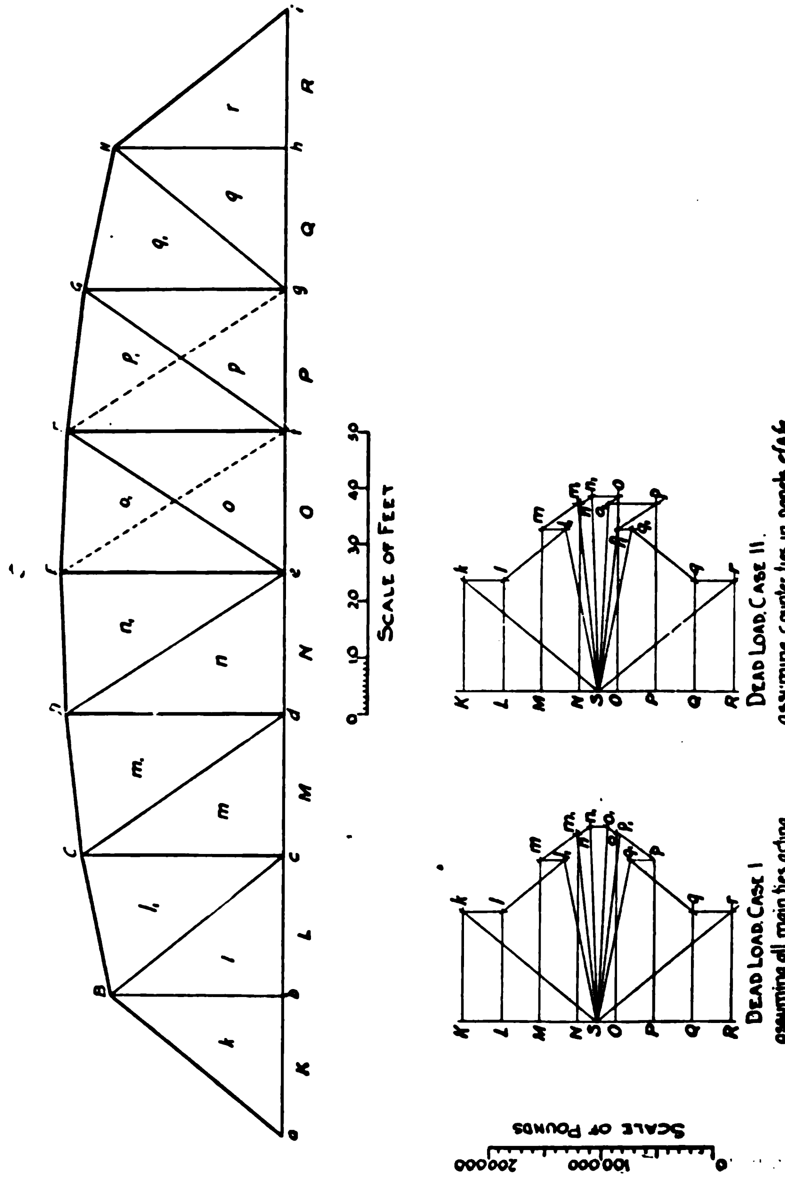


Fig. 14.—200-ft. Through Pratt Truss.

grams. The loads are taken from left to right, and laid off on the load-line from K to R downwards; then the right-hand reaction RS upwards; and, finally, the left-hand reaction SK to the point of beginning. In taking the forces at any panel-point in regular order from left to right, those acting away from the panel-point indicate tension in the corresponding member, and those acting towards it, compression.

Live-Load Stresses.

The live-load stresses will also be obtained graphically.

For the maximum shear in panel $a\,b$, which determines the stress in $a\,B$, as well as for the maximum moment at B, the position of the live-load is determined as in previous examples, viz.: the load in the panel to the left of b must be equal to or less than the total load on span divided by the number of panels in the span. With wheel 4 at b, as shown in Fig. 15, the load in panel $a\,b$ = 57,500 lbs., and the total load on span = 502,250 lbs. Then 57,500 is less than $\frac{502{,}250}{8}$ = 62,800. If wheel 5 were placed at b, it would be found that the load in the panel would be considerably greater than one-eighth of the total load on span. Thus the assumed position is correct. To obtain the panel-concentrations and reactions for this position of the live-load, verticals are dropped from the panel-points intersecting the equilibrium-polygon in points a_1, b_1, c_1, etc.; then lines are drawn through the pole in force-polygon, parallel with $a_1\,b_1$, $b_1\,c_1$, etc., and intersecting the load-line in points K, L, $M \ldots R$. The distances between these latter points represent the corresponding panel-concentrations. Another line drawn through the pole, parallel with the closing line $a_1\,i_1$, and intersecting the load line in the point S, determines the reactions $R\,S$ and $S\,K$. The stress-diagram is then proceeded with in the same manner as for dead-load stresses. Although this position of the load only gives the maximum stresses at end of truss, the stress-diagram is drawn for the whole truss, to illustrate the method more clearly.

On account of the slope of top chord, a portion of the shear in the intermediate panels is transmitted by it, and the remainder by the web-members. Therefore the general rule which

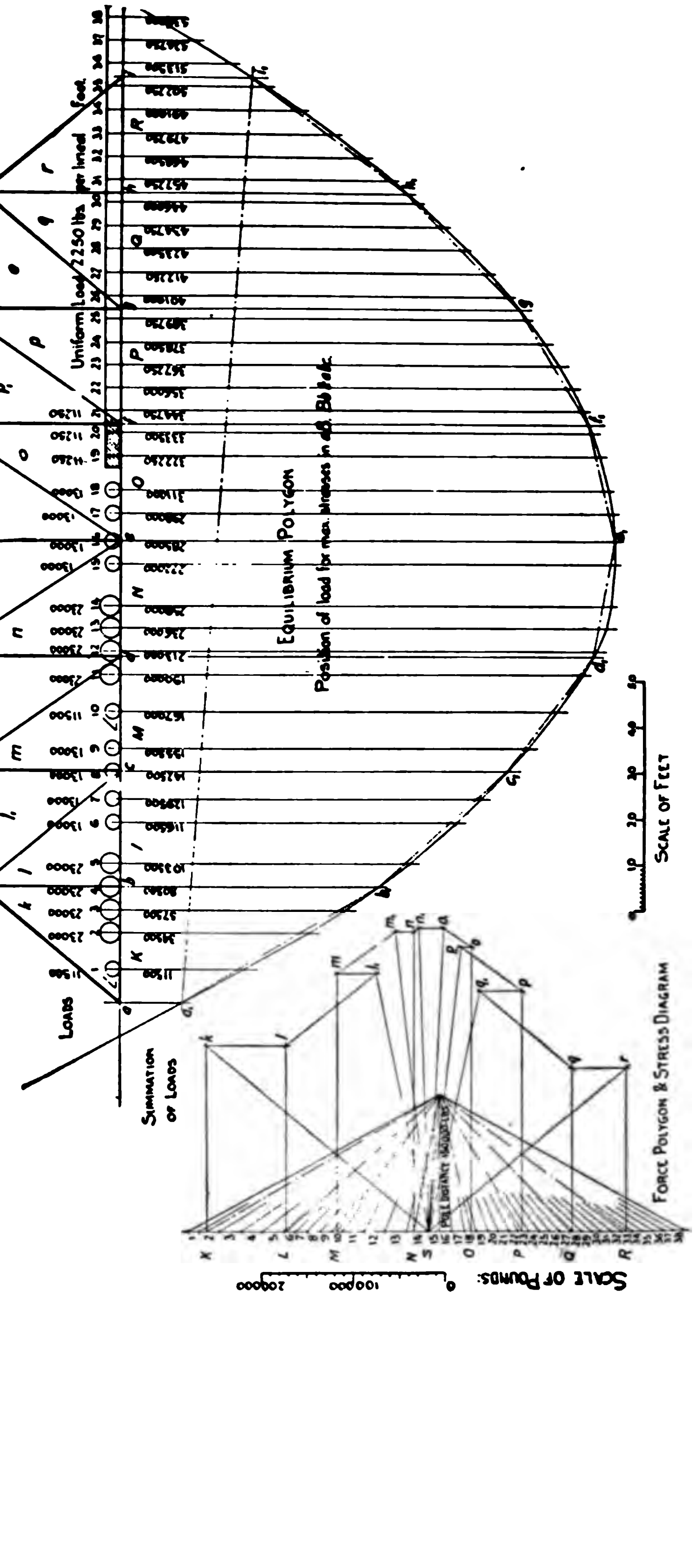


Fig. 15.—200-ft. Through Pratt Truss.

determines the position of the live-load for the maximum shear in a panel, does not apply strictly to the web-members in the panel; but, as will be seen later, it may be used in the majority of cases without appreciable error, especially when the slope of the top chord is not excessive, as in the present example.

For the maximum shear in panel $b\,c$, wheel 3 is placed at c, as shown in Fig. 16. The load in the panel to the left of c = 34,500 lbs., and the total load on span = 434,750 lbs. Then 34,500 is less than $\frac{434,750}{8} = 54,300$. With wheel 4 at c, the load in panel $b\,c$ would be greater than one-eighth of the total load on span. Thus the assumed position is correct. Now the position of live-load for maximum stress in $B\,c$ is determined by the following rule, the theory of which is fully explained in Johnson's "Theory and Practice of Modern Framed Structures":

$$G\left(1+\frac{x}{s}\right) \mathrel{\substack{=\\<}} \frac{W}{n}$$

in which W = the total load on span.
G = the total load in panel $b\,c$, not including load at c.
n = the number of panels in the truss.
x = the distance from end a to left-hand end of panel $b\,c$.
s = the distance from end a to the point where top-chord section $B\,C$, if produced, would intersect the bottom chord produced, which would be the point of moments for determining the stress in $B\,c$, if computed analytically.

It is found that s = 125 ft., and x = 25 ft. With wheel 3 at c, G = 34,500 lbs., and W = 434,750 lbs., n = 8. Then $34,500\left(1+\frac{25}{125}\right) = 40,400$, which is less than $\frac{434,750}{8} = 54,300$.

With wheel 4 at c, it is evident that $G\left(1+\frac{x}{s}\right)$ would be greater than $\frac{W}{n}$; thus the assumed position is correct, which is the

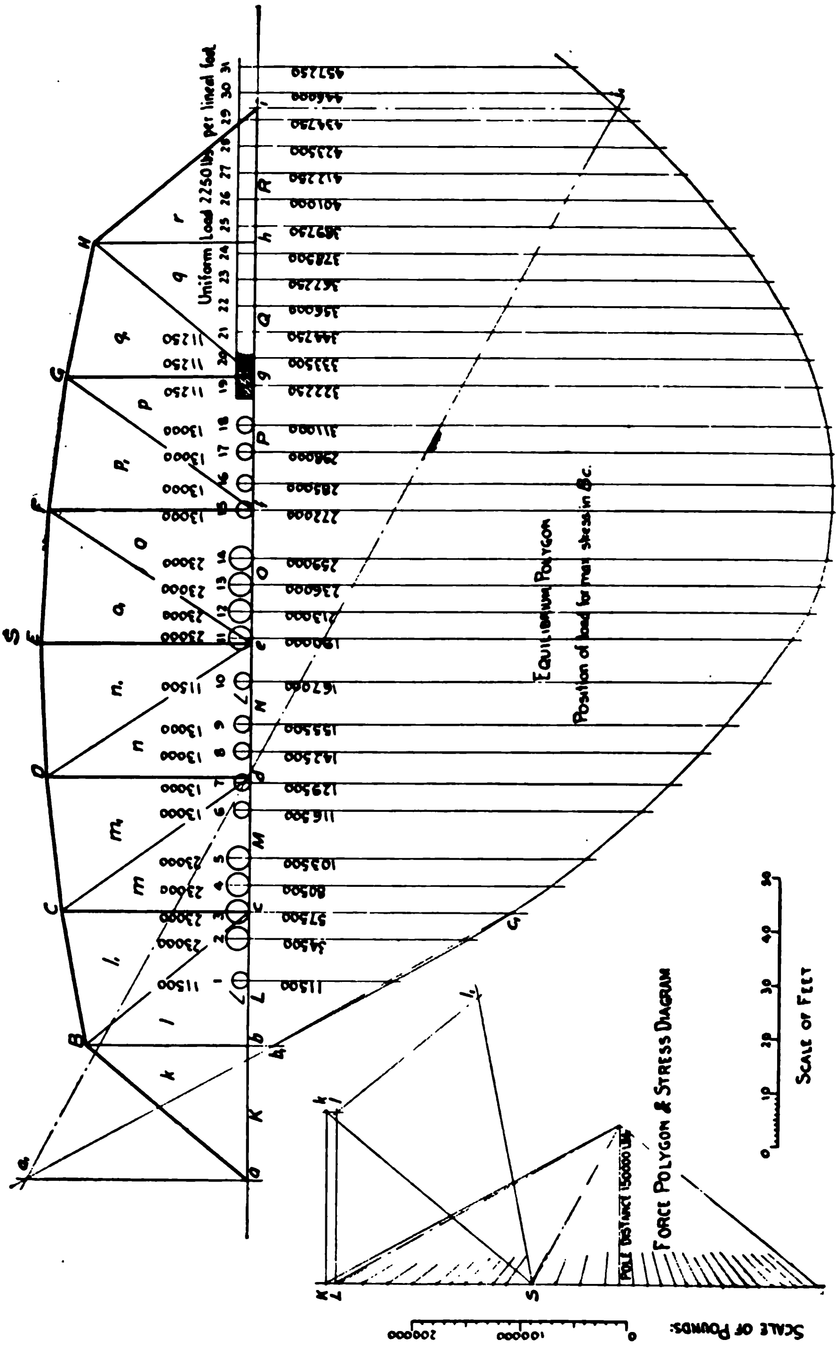


Fig. 16.—200-ft. Through Pratt Truss.

same as found by the rule for the maximum shear in panel $b\,c$. Verticals are drawn through a and i, intersecting the equilibrium-polygon in a_1 and i_1, through which latter points the closing line is drawn. Then a line drawn through the pole in force-polygon, parallel with closing line, and intersecting the load-line in S, determines the reaction $S\,K$. Since it is only intended to proceed with the stress-diagram far enough to obtain the stress in $B\,c$, it is unnecessary to obtain the panel concentrations, except the one ahead of the train, at b. Verticals are drawn through b and c, intersecting the equilibrium-polygon in b_1 and c_1; then, a line drawn through the pole in force-polygon, parallel with $b_1\,c_1$ and intersecting the vertical load-line in L, determines the concentration at b, which is $K\,L$. The stress-diagram is now drawn to obtain the stress in $B\,c$, which is $l\,l_1$ = 170,000 lbs.

For the maximum shear in panel $c\,d$, the load in the panel to the left of d should be equal to or less than one-eighth of the total load on span. With wheel 3 at d, the load in panel = 34,500 lbs., and the total load on span = 378,500 lbs. Then 34,500 is less than $\frac{378,500}{8} = 47,300$; and the assumed position is correct. If the chords were parallel, this position would give the maximum stresses in $C\,c$ and $C\,d$. The rule used in determining the position of load for the maximum stress in $B\,c$ will now be applied to $C\,c$ and $C\,d$. For $C\,c$, the point of moments for computing its stress analytically is at the point where the top-chord section $B\,C$, produced, intersects the bottom chord, produced. Then $s = 125$ ft. as before, and x is the distance from a to c, = 50 ft. Assuming wheel 3 at d, $G = 34,500$ lbs., and $W = 378,500$ lbs. Now, $34,500\left(1 + \frac{50}{125}\right) = 47,300$ is equal to $\frac{378,500}{8} = 47,300$. Thus the assumed position of load is correct for this member. For $C\,d$, the point of moments for computing its stress analytically is at the intersection of top-chord section $C\,D$, produced, with the bottom chord, produced. Thus s is found to be 242 ft.; $x = 50$ ft.; $G = 34,500$; $W = 378,500$. Then $34,500\left(1 + \frac{50}{242}\right) = 41,700$ lbs. is less than $\frac{378,500}{8} = 47,300$. Therefore wheel 3 at d

gives the maximum compression in $C\,c$ and the maximum tension in $C\,d$. Constructing the stress-diagram in the same manner as for the stress in $B\,c$, but continuing it a little farther, the maximum compression in $C\,c$ is found to be $+92{,}000$ lbs.; and the maximum tension in $C\,d$, $-120{,}000$ lbs.

Since the position of live-load for maximum shear has been found to give the maximum stresses in web-members in panels where the inclination of the top-chord is comparatively great, it may be assumed that it will give the maximum stresses in web-members near the center of span, where the top-chord is more nearly horizontal.

Then, for the maximum shear in panel $d\,e$, and the maximum stresses in $D\,d$ and $D\,e$, wheel 3 is placed at e, the load in panel $d\,e = 34{,}500$ lbs., and the total load on span $= 322{,}250$ lbs. Then 34,500 is less than $\frac{322{,}250}{8} = 40{,}300$. Ccnstructing the stress-diagram, it is found that the maximum compression in $D\,d = +63{,}000$ lbs., and the maximum tension in $D\,e = -91{,}000$ lbs.

With wheel 2 at f, the maximum compression in $E\,e = +43{,}000$ lbs., and the maximum tension in $E\,f = -68{,}000$ lbs.

With wheel 2 at g, the maximum tension in $F\,g = -43{,}000$ lbs.

The position of live-load for the maximum moment at any panel-point is determined as in the previous examples. That is to say, the moment at any panel-point is a maximum when the total load to the left of this point, divided by the number of panels between it and the left-hand abutment, is equal to or less than the total load on span divided by the number of panels in span. Furthermore, the live-load should cover the span, and the heavier wheel-concentrations should be brought as near as possible to the panel-point where the moment is required, with a wheel at the panel-point.

The maximum moment at c occurs with wheel 7 at this point; for the total load to the left of c divided by 2, is just less than the total load in span divided by 8. Thus $\frac{116{,}500}{2} = 58{,}250$ is less than $\frac{491{,}000}{8} = 61{,}375$. This position of the live-load

gives the maximum stress in $B\,C = +290{,}000$ lbs., and the maximum stress in $c\,d$, $= -285{,}000$ lbs.

The maximum moment at d occurs with wheel 11 at this point. The total load to the left of d = 167,000 lbs., and the total load on span = 491,000 lbs. Then $\frac{167{,}000}{3} = 55{,}700$ lbs. is less than $\frac{491{,}000}{8} = 61{,}375$ lbs. This position of the live-load gives the maximum stress in $C\,D = +330{,}000$ lbs., and the maximum stress in $d\,e = -328{,}000$ lbs.; also the maximum tension in $D\,d = -20{,}000$ lbs. These results are practically the same as those which may be scaled from the stress-diagram, Fig. 15.

The maximum moment at e occurs with wheel 13 at this point. The total load to the left of e = 213,000 lbs., and the total load on span = 457,250 lbs. Then, $\frac{213{,}000}{4} = 53{,}250$ lbs. is less than $\frac{457{,}250}{8} = 57{,}150$ lbs. This position of the live-load gives the maximum stress in $D\,E$ and $E\,F = +342{,}000$ lbs., and the maximum tension in $E\,e$, $= -27{,}000$ lbs.

Only one diagram of the truss is required, which should be drawn on a sheet of tracing cloth large enough to contain the stress-diagrams for all the different cases. The truss-diagram should be placed on the moment-diagram in the various positions stated above, and the reactions and panel-concentrations determined for each case. The various stress-diagrams may then be distributed over the sheet in any manner found most convenient.

The stresses, as already determined, are as follows:

Dead-Load Stresses (from Fig. 14).	Live-Load Stresses.
$a\,B = +153{,}000$	$a\,B = +314{,}000$ (wheel 4 at b)
$B\,b = -34{,}000$	$B\,b = -86{,}000$ (wheel 4 at b)
$B\,c = -72{,}000$	$B\,c = -170{,}000$ (wheel 3 at c)
$C\,c = +21{,}000$	$C\,c = +92{,}000$ (wheel 3 at d)
$C\,d = -40{,}000$	$C\,d = -120{,}000$ (wheel 3 at d)
$D\,d = -3{,}000$	$D\,d = +63{,}000$ (wheel 3 at e)
$D\,e = -10{,}000$	$D\,d = -20{,}000$ (wheel 11 at d)

Dead-Load Stresses (from Fig. 14).	Live-Load Stresses.
Ee = − 16,000 (Case I)	De = − 91,000 (wheel 3 at e)
Ee = − 22,000 (Case II)	Ee = − 27,000 (wheel 13 at e)
Ef = + 10,000 (Case II)	Ee = + 43,000 (wheel 2 at f)
Fg = + 40,000 (Case II)	Ef = − 68,000 (wheel 2 at f)
Bc = + 146,000	Fg = − 43,000 (wheel 2 at g)
CD = + 168,000	BC = + 290,000 (wheel 7 at c)
DE = + 172,000	CD = + 330,000 (wheel 11 at d)
abc = − 98,000	DE = + 342,000 (wheel 13 at e)
cd = − 143,000	abc = − 204,000 (wheel 4 at b)
de = − 167,000	cd = − 285,000 (wheel 7 at c)
	de = − 328,000 (wheel 11 at d)

The dead- and live-load stresses are shown in Fig. 13, with impact added according to specification.

In member Dd, the dead-load stress is − 3,000, and the live-load stresses are − 20,000 and + 63,000. Thus the total range of live-load stresses = 20,000 + 63,000 = 83,000, and the maximum stress which can occur at one time = 63,000 − 3,000 = 60,000. Then impact $= \frac{83{,}000^2}{60{,}000} = \pm 115{,}000$. In member Ee, the dead-load stress, Case I, = − 16,000, and the dead-load stress, Case II, = − 22,000. The live-load stress, − 27,000 occurs with Case I, and the live-load stress, + 43,000, occurs with Case II. The total range of live-load stresses = 27,000 + 43,000 = 70,000, and the maximum stress which can occur at one time = 16,000 + 27,000 = 43,000. Then impact $= \frac{70{,}000^2}{43{,}000}$ $= \pm 114{,}000$.

In the counter-tie Ef there is only a live-load stress, which is equal to that obtained with wheel 2 at f, less 0.7 of the dead-load stress (Case II) = 68,000 − 7,000 = 61,000; and the impact is equal to this net stress.

Likewise, in counter-tie Fg, the net live-load stress = − 43,000 + 30,000 = − 13,000; and the impact, as before, is equal to this net stress.

Laterals.

The top and bottom laterals are to be designed for a fixed horizontal force of 150 lbs. per lin. ft.; and the bottom laterals for a moving horizontal force of 400 lbs. per lin. ft.

The top-lateral system may be assumed to consist of a horizontal truss of 6 panels, supported horizontally at B and H by the portal struts. The bottom-lateral system consists of a horizontal truss of 8 panels. Length of panels = 25 ft. Depth of truss = 18 ft. Length of diagonals = $\sqrt{25^2+18^2}$ = 30.8 ft. Panel dead-load for top and bottom laterals = 150 lbs.x25 ft. = 3,750 lbs. Panel live-load for bottom laterals only = 400 lbs.x25 ft. = 10,000 lbs.

Top-Lateral Stresses.

SHEAR IN PANELS.	STRESS IN DIAGONALS.
$BC = 3{,}750 \times 2\frac{1}{2} = 9{,}375$	$1\text{st} = 9{,}375 \times \frac{30.8}{18} = -16{,}000$
$CD = 3{,}750 \times 1\frac{1}{2} = 5{,}625$	$2\text{d} = 5{,}625 \times$ " $= -9{,}600$
$DE = 3{,}750 \times \frac{1}{2} = 1{,}875$	$3\text{d} = 1{,}875 \times$ " $= -3{,}200$

Bottom-Lateral Stresses.

SHEAR IN PANELS.

$$ab = (3{,}750 \times 3\tfrac{1}{2}) + \left(10{,}000 \times \frac{28}{8}\right) = 48{,}125$$

$$bc = (3{,}750 \times 2\tfrac{1}{2}) + \left(10{,}000 \times \frac{21}{8}\right) = 35{,}625$$

$$cb = (3{,}750 \times 1\tfrac{1}{2}) + \left(10{,}000 \times \frac{15}{8}\right) = 24{,}375$$

$$de = (3{,}750 \times \tfrac{1}{2}) + \left(10{,}000 \times \frac{10}{8}\right) = 14\ 375$$

STRESS IN DIAGONALS.

$$1\text{st} = 48{,}125 \times \frac{30.8}{18} = -82{,}400$$

$$2\text{d} = 35{,}625 \times \text{"} = -60{,}900$$
$$3\text{d} = 24{,}375 \times \text{"} = -41{,}700$$
$$4\text{th} = 14{,}375 \times \text{"} = -24{,}600$$

Portal Strut.

The portal strut will be similar to that of the 150-ft. span, and the method of computing the stresses exactly the same.

The force applied at top = $3\frac{1}{2}$ panel loads of 3,750 lbs. = 13,125 lbs.

Horizontal reaction at foot of each post = $13{,}125 \times \frac{1}{2} = 6{,}560$ lbs.

The plane of contraflexure is assumed half-way between lower ends of posts and knee-connections.

Bending moment in posts at kneebrace connections = 6,560 lbs. × 16 ft. = 105,000 ft.-lbs. = 1,260,000 in.-lbs.

Forces required at top of posts to balance these moments = 105,000 × $\frac{1}{7}$ = 15,000 lbs.

Stress in top strut (windward side) = 15,000 + 13,125 = + 28,125 lbs.

Stress in top strut (leeward side) = — 15,000 lbs.

Horizontal forces at knee-connections = 15,000 + 6,560 = 21,560 lbs.

Stress in kneebraces = $21{,}560 \times \frac{11.4}{9}$ = - 27,200 lbs., for windward side; and + 27,200 lbs. for leeward side.

The sections required and material provided for laterals and portal struts are shown in Fig. 13.

PROPORTIONING OF TRUSS-MEMBERS.

The areas required for the tension members are determined directly by dividing the total stress in each member by the unit stress, 16,000 lbs. per sq. in.

The end-post *a B* must be designed so that the maximum fiber-stress, due to the combination of direct and bending stresses, shall not exceed that allowed for the direct stresses alone by more than 25%. Assuming

1 cover-plate	$24 \times \frac{1}{2}$	-in.	= 12.00
2 web-plates	$21 \times \frac{1}{2}$	"	= 21.00
4 angles	$6 \times 3\frac{1}{2} \times \frac{1}{2}$	"	= 18.00
			51.00 sq. ins.,

the radius of gyration with reference to an axis through the center of gravity and perpendicular to the cover-plate, is found to be 8.4 ins.; and the distance from this axis to the outer fibers = 12 ins.; $\frac{l}{r} = \frac{32}{8.4} = 3.8$, which from Table I, corresponds to a permissible unit stress of 14,350 lbs., for fixed ends. For the combination of direct and bending stresses, the permissible unit stress may be taken at 16,000 + 25% = 20,000 lbs.; for the maximum moments occur at foot of post and at knee-connection, at which points it is unnecessary to reduce the unit stress by a column formula.

The area required for direct compression alone = 678,000 ÷ 14,350 = 47.2 sq. ins.

The bending moment due to wind force, as already determined, = 1,260,000 in.-lbs.

Then, for combination of direct and bending stresses, the area required is as follows:

Area required for direct compression = 678,000 ÷ 20,000 = 33.90

Area required for bending moment (Chap. III) = $\frac{M\,n}{r^2 f}$ =

$$\frac{1{,}260{,}000 \times 12}{8.4^2 \times 20{,}000} = \qquad 11.50$$

sq. ins. 45.40

The first condition requires the greater area, and the following section will be used:

1 cover-plate	$24 \times \frac{1}{2}$	-in.	= 12.00
2 web-plates	$21 \times \frac{7}{16}$	"	= 18.38
4 angles	$6 \times 3\frac{1}{2} \times \frac{1}{2}$	"	= 18.00
			48.38 sq. ins.

For the top chord the same form of section will be used. The least radius of gyration is about an axis through the center of gravity and parallel with the cover-plate. It is found to be 7.9 ins.

Top Chord B C.—Length = 25.5 ft.; $\frac{l}{r} = \frac{25.5}{7.9} = 3.2$; permissible unit stress for fixed ends (by Table I) = 14,800 lbs.; total stress = 629,000 lbs. Then 629,000 ÷ 14,800 = 42.5 sq. ins. required. The specification requires that the unsupported width of cover-plates for top chords and end-posts shall not exceed 40 times their thickness (because material of less relative thickness is not considered efficient in compression); and this unsupported width, or distance between rows of rivets, in the present example = 24 − 3 = 21 ins. Then the minimum thickness of cover-plates permissible = $\frac{21}{40}$ = ½-in. The following section will be used for BC:

1 cover-plate	$24 \times \frac{1}{2}$	in.	= 12.00
2 web-plates	$21 \times \frac{3}{8}$	"	= 15.75
4 angles	$6 \times 3\frac{1}{2} \times \frac{7}{16}$	"	= 15.88
			43.63 sq. ins.

The sections required and those used for members CD and DE are shown in Fig. 13.

Bottom Chords.—The bottom chords will be made of four 8x4½-in. angles, and, where necessary, side-plates will be added. For the net area, allowance will be made for two holes, 1 in. in diameter, in each angle and each plate. The areas required, as well as the material provided, are given in Fig. 13.

Intermediate Posts.—For the intermediate posts four bulb-angles will be used, as shown. Carnegie section " B 132," the dimensions of which are 8x3½x⅝-in., will be tried. The area of one angle = 5.66 sq. ins., and its moment of inertia about an axis through its center of gravity and parallel with the shorter leg = 48.8. The position of its center of gravity is not given in the handbook, but may be found readily by drawing the section to scale on stiff paper, cutting it out and then balancing on a knife-edge. It is located 3.82 ins. from the back of shorter leg. The 3½-in. legs will be separated $\frac{5}{16}$-in., which is the thickness of web-plate. Then the distance from the neutral axis of the whole section to the center of gravity of the angles will be $3.82 + 0.16 = 3.98$ ins. The web-plates are not noted on the stress-diagram, as they are not assumed to be part of the effective section of the posts. The moment of inertia of the four angles about an axis through the center of gravity of the whole figure and parallel with web-plate =

$$I = (4 \times 5.66 \times 3.98^2) + (4 \times 48.8) = 553.8.$$

$$r = \sqrt{\frac{553.8}{22.64}} = 4.94 \text{ ins.}$$

The total stress in Cc, including impact, = 188,000 lbs.; length = 35 ft.; $\frac{l}{r} = \frac{35}{4.94} = 7.1$, which corresponds to a unit stress of 11,400 lbs. for fixed ends. Then $188{,}000 \div 11{,}400 = 16.49$ sq. ins. required. The area of the assumed section, 22.64 sq. ins., is larger than required for the stress; but the short leg of a 7-in. bulb-angle, which measures but 3 ins., does not give sufficient room for driving the rivets in web-plate; and the radius of gyration of a post composed of four plain 7x3½-in. angles would not be great enough to fulfil the condition that the length of post shall not exceed 100 times its least radius of gyration.

The same material will be used for posts Dd and Ee.

Stringers.

The stringers will be exactly the same as those of the 150-ft. span. The calculations for them will not be repeated.

Intermediate Floorbeams.

Effective length = 18 ft.; stringer-concentrations 8 ft. apart and 5 ft. from center of trusses; depth of web-plate = 4 ft. 6 ins.; effective depth of floorbeam 4.25 ft.; assumed weight of floorbeam = 3,000 lbs.

The dead-load concentrations from stringers = 450 lbs. × 25 ft. = 11, 250 lbs. The live-load concentrations from stringers = 86,000 lbs.

END SHEAR:

Dead-load	$= (3{,}000 \times \frac{1}{2}) + 11{,}250$	= 12,750
Live-load		= 86,000
Impact	$= \dfrac{86{,}000^2}{86{,}000 + 12{,}750}$	= 74,900
		173,650 lbs.

Area required in web-plate = 173,650 ÷ 10,000 = 17.36 sq. ins. A 54x$\frac{3}{8}$-in. web-plate = 20.25 sq. ins., will be used.

MOMENT:

Dead-load	$= \dfrac{3{,}000 \times 18}{8} + (11{,}250 \times 5)$	= 63,000
Live-load	$= 86{,}000 \times 5$	= 430,000
Impact	$= \dfrac{430{,}000^2}{430{,}000 + 63{,}000}$	= 374,000
		867,000 ft.-lbs.

Flange-stress = 867,000 ÷ 4.25 ft. = 204,000 lbs.
Flange-area required = 204,000 ÷ 16,000 = 12.75 sq. ins.

Area provided: $\frac{1}{8}$ of 54x$\frac{3}{8}$-in. web-plate	= 2.53	
2 angles, 6x6x-$\frac{9}{16}$in.	= 10.61	(2 holes. 1 in. in diam. in each angle.)
	13.14 sq. ins., net.	

End Floorbeams.

The effective length, location of stringer-concentrations, depth of web-plate, and assumed weight of end floorbeams are the same as for intermediate floorbeams. As a single line of rivets will be

found sufficient to transmit the longitudinal shearing stresses from the web-plate to the flanges, 6x3½-in. angles will be used for the latter, with the 3½-in. legs vertical; and the effective depth will be 4.33 ft. The dead-load concentrations from stringers = 450 lbs. × 12.5 ft. = 5,600 lbs. The live-load concentrations from stringers = 64,900 lbs.

END SHEAR:

$$\text{Dead-load} = (3{,}000 \times \tfrac{1}{2}) + 5{,}600 = 7{,}100$$

$$\text{Live-load} = 64{,}900$$

$$\text{Impact} = \frac{64{,}900^2}{64{,}900 + 7{,}100} = \underline{58{,}500}$$

$$130{,}500 \text{ lbs.}$$

Area required in web-plate = 130,500 ÷ 10,000 = 13.05 sq. ins. A 54x⅜-in. web-plate = 20.25 sq. ins., will be used.

MOMENT:

$$\text{Dead-load} = \frac{3{,}000 \times 18}{8} + (5{,}600 \times 5) = 34{,}750$$

$$\text{Live-load} = 64{,}900 \times 5 = 324{,}500$$

$$\text{Impact} = \frac{324{,}500^2}{324{,}500 + 34{,}750} = \underline{293{,}000}$$

$$652{,}250 \text{ ft.-lbs.}$$

Flange-stress = 652,250 ÷ 4.33 = 150,500 lbs.
Flange-area required = 150,500 ÷ 16,000 = 9.4 sq. ins.

Area provided: ⅛ of 54x⅜-in. web-plate = 2.53
2 angles, 6x3½x$\frac{7}{16}$-in. = 7.06 (1 hole 1 in. in diam. in each angle.)

9.59 sq. ins., net.

Estimated Weight.

Two Trusses.

$$aB = 48.38 \text{ sq. ins.} \times 39.0 \text{ ft.} \times \frac{10}{3} \times 4 = 25{,}200$$

$$BC = 43.63 \text{ " } \times 25.5 \text{ " } \times \frac{10}{3} \times 4 = 14{,}800$$

$$CD = 48.38 \text{ " } \times 25.3 \text{ " } \times \frac{10}{3} \times 4 = \underline{16{,}300}$$

Forward 56,300

			Brought forward	56,300
DE	=51.00 sq. ins.	x25. ft.	$x\frac{10}{3}x4=$	17,000
abc	=35.16 "	x25. "	$x\frac{10}{3}x8=$	23,400
cd	=45.36 "	x25. "	$x\frac{10}{3}x4=$	15,100
de	=53.72 "	x25. "	$x\frac{10}{3}x4=$	17,900
Bb	=17.60 "	x30. "	$x\frac{10}{3}x4=$	7,000
Cc	=22.64 "	x35. "	$x\frac{10}{3}x4=$	10,600
Dd	=22.64 "	x38. "	$x\frac{10}{3}x4=$	11,500
Ee	=22.64 "	x39. "	$x\frac{10}{3}x2=$	5,900
Struts	= 6.84 "	x19.5 "	$x\frac{10}{3}x4=$	1,800
Bc	=29.44 "	x39. "	$x\frac{10}{3}x4=$	15,300
Cd	=21.32 "	x43. "	$x\frac{10}{3}x4=$	12,200
De	=19.12 "	x 45.5 "	$x\frac{10}{3}x4=$	11,600
Ef	=10.06 "	x46.2 "	$x\frac{10}{3}x4=$	6,500
Fg	= 4.96 "	x45.5 "	$x\frac{10}{3}x4=$	3,000
				215,100
Details, (30%)			=	64,500
				279,600 lbs.

The weights of floorbeams, stringers, portal struts, and stringer laterals will be practically the same as those of the 150-ft. span.

Top Laterals.

	24 angles	3x3 x$\frac{5}{16}$-in.	@ 6.10 lbs.	27 ft. long	=	3,960
Gussets.	48 plates	12x$\frac{5}{16}$	" @ 12.75 "	1 " "	=	610
Tie-plates.	12 plates	12x$\frac{5}{16}$	" @ 12.75 "	1.7 " "	=	780
Splices.	12 plates	12x$\frac{5}{16}$	" @ 12.75 "	1 " "	=	150
Latticing.	300 flats	2½x$\frac{5}{16}$	" @ 2.65 "	2 " "	=	1,590
						7,090
Rivet-heads, (3%)					=	210
						7,300

Intermediate Struts and Sway-Bracing.

Top struts.	20 angles	3½x3 x$\frac{5}{16}$-in.	@ 6.60 lbs.	16 ft. long	=	2,140
Lower struts.	10 angles	3½x3 x$\frac{5}{16}$	" @ 6.60 "	16 " "	=	1,060
Diagonals.	10 angles	3x3 x$\frac{5}{16}$	" @ 6.10 "	abt. 20 " "	=	1,220
Verticals.	5 angles	3x3 x$\frac{5}{16}$	" @ 6.10 "	5 " "	=	150
Gussets.	10 plates	18x ⅜	" @ 22.96 "	2 " "	=	460
Latticing.	60 flats	2½x$\frac{5}{16}$	" @ 2.65 "	2 " "	=	320
						5,350
Rivet-heads, (3%)					=	150
						5,500

Bottom Laterals.

1st diagonals	4 angles	8x4½x$\frac{9}{16}$-in.	@ 22.80 lbs.	27 ft. long	=	2,470
2d diagonals	4 angles	6x3½x$\frac{9}{16}$	" @ 15.30 "	27 " "	=	1,650
3d diagonals	4 angles	6x3½x ⅜	" @ 11.70 "	27 " "	=	1,260
4th diagonals	4 angles	3½x3½x ⅜	" @ 8.50 "	27 " "	=	920
						6,300
Details, (33⅓%)					=	2,100
						8,400

Summary.

2 Trusses		= 279,600
2 End Floorbeams	@ 3,200	= 6,400
7 Intermediate Floorbeams	@ 3,590	= 25,130
16 Stringers	@ 4,270	= 68,320
8 Panels Stringer-Bracing	@ 290	= 2,320
Bottom Laterals		= 8,400
Top Laterals		= 7,300
2 Portal Struts	@ 1,510	= 3,020
5 Intermediate Struts		= 5,500
Pier-Members		= 20,000
Total,		425,990 lbs

The weight of steel, exclusive of pier-members, = 405,990 lbs. Then 405,990 ÷ 200 = 2,030 lbs. per lin. ft., which is 70 lbs. less than weight of steel assumed for computation of stresses.

Details.

The details generally will be similar to those of the 150-ft. span. It is only necessary to add that the top-chord members at *C*, *D*, *E*, *F* and *G* will be faced to give perfect bearings; but, in addition, sufficient rivets should be provided in the splice-plates at these points for about one-half the stresses.

CHAPTER VI.

THE DESIGN OF A 170-FT. SWING-BRIDGE.

Length, 170 ft. c. to c. of end-bearings = 8 panels of 20 ft. and one panel of 10 ft.

Depth, 28 ft. c. to c. of chords.

Length of diagonals $= \sqrt{20^2+28^2} = 34.41$ ft.

Width, 16 ft. clear and 17 ft. 6 ins. c. to c. of trusses.

Dead-load:	floor,	600
	steel,	1,300
	total,	1,900 lbs. per lin. ft.

Live-load as per specification.

For the trusses and loading girders of swing-spans, it is usually customary to reduce the wheel-loads to equivalent uniform loads per lin. ft., because of the great amount of labor in figuring the stresses for wheel-concentrations. A sufficiently satisfactory method of doing this is to divide the maximum load on a given length by this length. On one arm of the bridge under consideration it is possible to place the whole of the forward engine and five wheels of the second engine, making a total weight of 518,000 lbs. Then $518,000 \div 80 = 6,475$ lbs. per lin. ft. when one arm only is loaded. On the whole bridge both engines may be placed, and, in addition, 61 ft. of the specified uniform load, making a total load of 896,500 lbs. Then $896,500 \div 170 = 5,273$ lbs. per lin. ft. when the bridge is loaded over all. The assumed live-loads are as follows:

When one arm only is loaded, 6,500 lbs. per lin. ft.

When bridge is loaded over all, 5,300 lbs. per lin. ft.

$$\text{Panel dead-load for one truss} = \frac{1,900}{2} \times 20 \text{ ft.} = 19,000 \text{ lbs.}$$

The dead-load concentrations at end panel-points will be assumed equal to three-fourths of the intermediate panel dead-loads = 14,250 lbs.

Panel live-load for one truss (one arm loaded) $= \frac{6500}{2} \times 20$ $= 65,000$ lbs.

Panel live-load for one truss (bridge loaded over all) $= \frac{5,300}{2} \times 20 = 53,000$ lbs.

There are three cases to be considered for stresses, as follows:

Case I, dead-load, bridge swinging; or closed, with ends barely touching supports.

Case II, dead-load, bridge closed, ends lifted so that bottom chord at all points of support will lie in the same horizontal plane.

Case III, live-load, bridge closed, ends lifted as before.

Case I will first be considered.

Shear in panel $a\,b$ is equal to load at a.

Shear in panel $b\,c$ is equal to loads at a and b.

Shear in panel $c\,d$ is equal to loads at a, b and c.

Shear in panel $d\,e$ is equal to loads at a, b, c and d.

Stress in $a\,b\,c$ is equal to moment of load a about panel-point B, divided by depth of truss.

Stress in $B\,C\,D$ is equal to moments of loads a and b about panel-point c, divided by depth of truss.

Stress in $c\,d\,e$ is equal to moments of loads a, b and c about panel-point D, divided by depth of truss.

Stress in $D\,E\,F\,G$ and $e\,f$ is equal to moments of loads a, b, c and d about panel-point e, divided by depth of truss.

STRESSES—CASE I

B [illegible] d			$= -\ 19,000$
C [illegible] e			$= 0$
$a\,B$	$= 14,250$	$\times \frac{34.41}{28}$	$= -\ 17,500$
$B\,c$	$= (14,250 + 19,000) \times \frac{34.41}{28}$		$= +\ 40,800$
$c\,D$	$= \left[14,250 + (19,000 \times 2)\right] \times \frac{34.41}{28}$		$= -\ 64,200$
$D\,e$	$= \left[14,250 + (19,000 \times 3)\right] \times \frac{34.41}{28}$		$= +\ 87,500$
$a\,b\,c$	$= 14,250 \times 20 \times \frac{1}{28}$		$= +\ 10,300$

$$BCD = \left[(14{,}250 \times 40) + (19{,}000 \times 20)\right] \times \frac{1}{28} = -34{,}000$$

$$cde = \left[(14{,}250 \times 60) + (19{,}000 \times 40) + (19{,}000 \times 20)\right] \times \frac{1}{28} = +71{,}500$$

$$DEFG = \left[(14{,}250 \times 80) + (19{,}000 \times 60) + (19{,}000 \times 40) + (19{,}000 \times 20)\right] \times \frac{1}{28} = -122{,}000$$

$$ef = DEFG = +122{,}000$$

Before proceeding with Cases II and III, it will be necessary to obtain coefficients for the end-reactions. The truss is assumed to be only partially continuous, or incapable of transferring shear across the center panel. The light diagonals used in this panel are required only to prevent the bridge from tipping when open, which otherwise might happen if there should be an unbalanced load on one end.

The following formulas* are used to obtain the reactions for partially continuous t—sses of two equal end spans and a short center span.

$$R_1 = P\left(1 - k\right) - \left[\frac{P}{4 + 6\ n}\ (k - k^3)\right]$$

$$R_2 = P\ k + \left[\frac{P}{4 + 6\ n}\ (k - k^3)\right]$$

$$R_3 = \frac{P}{4 + 6\ n}\ (k - k^3)$$

$$R_4 = -\frac{P}{4 + 6\ n}\ (k - k^3)$$

in which l = length of either end span.
n = length of center span in terms of l.
k = distance of load P from outer end of span in which it is situated, in terms of l.
P = a concentrated load.

* See Merriman and Jacoby's "Higher Structures" (Part IV of "Roofs and Bridges").

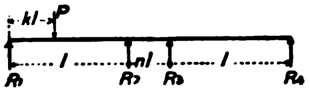


FIG. 18.

Then, referring to Figs. 17 and 18,

$l = 80$ ft.; $nl = 10$ ft. $\therefore n = \frac{1}{8} = 0.125$.

$k = \frac{1}{4} = 0.25$; $k^3 = 0.016$, for load at panel-point b.

$k = \frac{1}{2} = 0.5$; $k^3 = 0.125$, for load at panel-point c.

$k = \frac{3}{4} = 0.75$; $k^3 = 0.422$, for load at panel-point d.

Only the reactions R_1 and R_4 will be required, which will be computed for a load of 1 lb. at panel-points b, c and d.

$$\text{For load at } b, R_1 = (1 - 0.25) - \left[\frac{1}{4 + (6 \times 0.125)}(0.25 - 0.016)\right] = +0.701$$

$$R_4 = -\frac{1}{4 + (6 \times 0.125)}(0.25 - 0.016) = -0.049$$

$$\text{For load at } c, R_1 = (1 - 0.5) - \left[\frac{1}{4 + (6 \times 0.125)}(0.5 - 0.125)\right] = +0.421$$

$$R_4 = -\frac{1}{4 + (6 \times 0.125)}(0.5 - 0.125) = -0.079$$

$$\text{For load at } d, R_1 = (1 - 0.75) - \left[\frac{1}{4 + (6 \times 0.125)}(0.75 - 0.422)\right] = +0.181$$

$$R_4 = -\frac{1}{4 + (6 \times 0.125)}(0.75 - 0.422) = -0.069$$

By combining the above results, the coefficients for end-reactions with various dispositions of load are as follows:

For load at b only, $R_1 = +0.701$.

For loads at b and c, $R_1 = +0.701 + 0.421 = +1.122$.

For loads at b, c, and d, $R_1 = +0.701 + 0.421 + 0.181 = +1.303$.

For loads at c and d, $R_1 = +0.421 + 0.181 = +0.602$.

For loads at all points, $R_1 = +0.701 + 0.421 + 0.181 - 0.049 - 0.079 - 0.069 = +1.106$.

The actual reactions at a are obtained by multiplying the panel dead- and live-loads by the above coefficients.

REACTIONS FOR CASES II AND III.

Case II;	Loads at *b*, *c*, *d*, *g*, *h*, & *i*,	$R_1 = 19{,}000 \times 1.106 = 21{,}000$
Case III;	Load at *b* only,	$R_1 = 65{,}000 \times 0.701 = 45{,}600$
"	Loads at *b* & *c*,	$R_1 = 65{,}000 \times 1.122 = 72{,}900$
"	Loads at *b*, *c* & *d*,	$R_1 = 65{,}000 \times 1.303 = 84{,}700$
"	Loads at *c* & *d*,	$R_1 = 65{,}000 \times 0.602 = 39{,}100$
"	Loads at *b*, *c*, *d*, *g*, *h* & *i*,	$R_1 = 53{,}000 \times 1.106 = 58{,}600$

Shear in panel *a b* is equal to reaction *a*.

Shear in panel *b c* is equal to reaction *a*, less the load at *b*.

Shear in panel *c d* is equal to reaction *a*, less the loads at *b* and *c*.

Shear in panel *d e* is equal to reaction *a*, less the loads at *b*, *c* and *d*.

Stress in *a b c* is equal to the moment of reaction *a* about panel-point *B*, divided by depth of truss.

Stress in *B C D* is equal to the moment of reaction *a* about panel-point *c*, less the moment of load at *b* about panel-point *c*, divided by depth of truss.

Stress in *c d e* is equal to the moment of reaction *a* about panel-point *D*, less the moments of loads at *b* and *c* about panel-point *D*, divided by depth of truss.

Stress in *D E F G* and *e f* is equal to moment of reaction *a* about panel-point *e*, less the moments of loads at *b*, *c*, and *d* about panel-point *e*, divided by depth of truss.

b STRESSES—CASE II.

B d and *D d*		$= -19{,}000$
C c and *E e*		$= 0$
a B	$= 21{,}000 \times \frac{34.41}{28}$	$= +25{,}800$
B c	$= (21{,}000 - 19{,}000) \times \frac{34.41}{28}$	$= -\ 2{,}500$
c D	$= \left[21{,}000 - (19{,}000 \times 2) \times\right] \frac{34.41}{28}$	$= -20{,}800$
D e	$= \left[21{,}000 - (19{,}000 \times 3)\right] \times \frac{34.41}{28}$	$= +44{,}300$
a b c	$= 21{,}000 \times 20 \times \frac{1}{28}$	$= -15{,}000$
B C D	$= \left[(21{,}000 \times 40) - (19{,}000 \times 20)\right] \times \frac{1}{28}$	$= +16{,}400$

$$c\,d\,e \quad = \left[(21{,}000\times 60) - (19{,}000\times 40) - (19{,}000\times 20)\right]\times\frac{1}{28} = -\ 4{,}200$$

$$D\,E\,F\,G = \left[(21{,}000\times 80) - (19{,}000\times 60) - (19{,}000\times 40) \cdot (19{,}000\times 20)\right]\times\frac{1}{28} = -21{,}400$$

$$e\,f \quad = \left[(21{,}000\times 80) - (19{,}000\times 60) - (19{,}000\times 40) - (19{,}00\times 20)\right]\times\frac{1}{28} = +21{,}400$$

When the moment at any point is positive, the stress in top chord opposite this point is compression and in the bottom chord tension; and when the moment at any point is negative, the stress in top chord opposite this point is tension and in the bottom chord compression.

Position of Live-Load for Maximum Stresses.

A load at *b* only will give the maximum compression in *B c*.

Loads at *b* and *c* will give the maximum tension in *c D*.

Loads at *b*, *c* and *d* will give the maximum compression in top chord *B C D* and the maximum tension in bottom chord *a b c d e*; also the maximum compression in *a B* and *D e*.

Loads at *c* and *d* will give the maximum tension in *B c*.

Loads at *b*, *c*, *d*, *g*, *h* and *i* will give the maximum tension in top chord *D E F G*, and the maximum compression in bottom chord *e f*.

Loads at *a*, *e*, *f* and *j* will not affect the stresses in truss.

The reactions for the various dispositions of live-load have already been determined.

Stresses—Case III.

$$a\,B \quad = 84{,}700\times\frac{34.41}{28} = +\,104{,}200 \text{ (Loads at } b, c, \& \ d)$$

$$B\,c \quad = (45{,}600 - 65{,}000)\times\frac{34.41}{28} = +\ 23{,}800 \text{ (Load at } b \text{ only)}$$

$$B\,c \quad = 39{,}000\times\frac{34.41}{28} = -\ 48{,}100 \text{ (Loads at } c \ \& \ d)$$

$cD \quad = \left[72{,}900 - (65{,}000 \times 2)\right] \times \frac{34.41}{28} = -\ 70{,}200$ (Loads at b & c)

$De \quad = \left[84{,}700 - (65{,}000 \times 3)\right] \times \frac{34.41}{28} = +135{,}700$ (Loads at b, c & d)

$abc \quad = 84{,}700 \times 20 \times \frac{1}{28} \qquad = -60{,}500$ (Loads at b, c & d)

$BCD \quad = \left[(84{,}700 \times 40) - (65{,}000 \times 20)\right] \times \frac{1}{28}.$

$= +74{,}600$ (Loads at b, c & d)

$cde \quad = \left[(84{,}700 \times 60) - (65{,}000 \times 40) - (65{,}000 \times 20)\right] \times \frac{1}{28} = -42{,}200$ (Loads at b, c & d)

$DEFG = \left[(58{,}600 \times 80) - (53{,}000 \times 60) - (53{,}000 \times 40) - (53{,}000 \times 20)\right] \times \frac{1}{28}$

$= -59{,}900$ (Loads at $b, c, d, g, h,$ & i)

$ef \quad = \left[(58{,}600 \times 80) - (53{,}000 \times 60) - (53{,}000 \times 40) - (53{,}000 \times 20)\right] \times \frac{1}{28}$

$= +\ 59{,}900$ (Loads at $b, c, d, g, h,$ & i)

The live-load stress in the vertical hangers Bb and Dd = $-74{,}700$. It is obtained from the actual wheel-loads, placing wheel No. 4 at b or d. The stresses are summarized on the stress-diagram, Fig. 17. Case III is combined either with Case I or Case II; for, if the ends of the bridge are barely touching the supports, Case I will represent the dead-load stresses; but if the ends are lifted sufficiently, Case II will represent the dead-load stresses. The correct dead-load stresses will always be somewhere between these two extremes.

For a truss with sloping chords the reactions are usually computed in the same manner as given in the present example, and the stresses obtained graphically. Separate stress-diagrams are constructed for Cases I and II, and one for each position of live-load, Case III.

IMPACT.

As stated in the specification, when the live-load and dead-load stresses are of the same kind, the impact shall be computed by squaring the live-load stress and dividing by the live-load plus the dead-load stress; but, when a member is subject to alternate live-load stresses, the impact shall be computed by squaring the total range of live-load stress and dividing by the maximum stress which can occur at one time.

For member $a\,B$, the dead-load stress, Case II, $= +25{,}800$, and the live-load stress $= +104{,}200$. Impact $= \dfrac{104{,}200^2}{104{,}200 + 25{,}800}$ $= +83{,}000$.

For member $B\,C\,D$, the dead-load stress, Case II $= +16{,}400$, and the live-load stress $= +74{,}600$. Impact $= \dfrac{74{,}600^2}{74{,}600 + 16{,}400}$ $= +61{,}000$.

For member $D\,E\,F\,G$, the dead-load stress, Case I $= -\ 122{,}400$, and the live-load stress $= -\,59{,}900$. Impact $= \dfrac{59{,}900^2}{59{,}900 + 122{,}400}$ $= -19{,}800$.

For member $a\,b\,c$, the dead-load stress, Case II $= -\,15{,}000$, and the live-load stress $= -\,60{,}500$. Impact $= \dfrac{60{,}500^2}{60{,}500 + 15{,}000}$ $= -48{,}500$.

For member $c\,d\,e$, the dead-load stress, Case I $= +71{,}500$, and the live-load stress $= -42{,}200$. Here the total range of live-load stress $= 42{,}200$, and the maximum stress which can occur at any time $= 71{,}500$. Thus the impact to be added to Case I $= \dfrac{42{,}200^2}{71{,}500} = +25{,}000$.

For the same member, the dead-load stress, Case II $= -\,4{,}200$, and the live-load stress, as before $= -42{,}200$. Then the impact to be added to Case II and the live-load $= \dfrac{42{,}200^2}{42{,}200 + 4{,}200}$ $= -\,38{,}500$.

For member $B\ c$, the dead-load stress, Case I = +40,800, and the live-load stresses = +23,800 and −48,100. Then the total range of live-load stresses = 23,800 + 48,100 = 71,900, and the maximum stress which can occur at any time = 40,800 + 23,800 = 64,600. Impact to be used with Case I and the live-load compression $= \frac{71,900^2}{64,600} = +80,000.$

For the same member, the dead-load stress, Case II = −2,500 and the live-load stresses, as before = +23,800 and −48,100. The total range of live-load stresses = 71,900, and the maximum stress = 2,500 + 48,100 = 50,600. Impact to be used with Case II and the live load tension $= \frac{71,900^2}{50,600} = -102,000.$

For member $c\ D$, the dead-load stress, Case I = − 64,300, and the live-load stress = − 70,200. Impact $= \frac{70,200^2}{70,200+64,300}$ = −36,700.

For member $D\ e$, the dead-load stress, Case I = +87,000, and the live-load stress = +135,700. Impact $= \frac{135,700^2}{135,700+87,000}$ = +82,500.

For members $B\ b$ and $D\ d$, the dead-load stress = −19,000, and the live-load stress = −74,700. Impact $= \frac{74,700^2}{74,700+19,000}$ = −59,700.

The impacts are combined with the other stresses in Fig. 17, and the sections required and provided are shown.

The 18 x $\frac{5}{16}$-in. cover-plates on end-posts, and the 11 x $\frac{5}{16}$-in. web-plates in the vertical members are not counted on as effective section, but as tie-plates only.

STRINGERS FOR 20-FT. PANELS.

Dead-load $= \frac{600}{2} + 150$ = 450 lbs. per lin. ft.

The live-load consists of four driving-wheel loads of 23,000 lbs. each, spaced 5- ft. centers.

The maximum live-load shear is obtained by placing the first driving wheel at one end of stringer; then taking moments of the four loads about the opposite end, and dividing by the span. The maximum live-load moment will be computed by the formula $M = P\left(l-2a+\frac{a^2}{4l}\right)$, in which P = one load, l = length of span, a = distance between loads.

SHEAR:

Dead-load	=	450×10 =	4,500
Live-load	=	$23{,}000 \times \frac{50}{20}$ =	57,500
Impact	=	$\frac{57{,}500^2}{57{,}500+4{,}500}$ =	53,300
			115,300 lbs.

Area required in web-plate = 115,300 ÷ 10,000 = 11.53 sq. ins.

A 36 x ⅜-in. web-plate, = 13.5 sq. ins., will be used.

MOMENT:

Dead-load	=	$\frac{450 \times 20^2}{8}$ =	22,500
Live-load	=	$23{,}000\left(20-10+\frac{25}{80}\right)$ =	237,200
Impact	=	$\frac{237{,}200^2}{237{,}200+22{,}500}$ =	216,600
			476,300 ft.-lbs.

Effective depth of stringer = 2.75 ft.
Flange-stress = 476,300 ÷ 2.75 = 173,200 lbs.
Flange-area required = 173,200 ÷ 16,000 = 10.82 sq. ins.
Then, ⅛ of 36 x ⅜-in. web-plate = 1.69
2 angles 6 x 6 x $\frac{7}{16}$-in. = 9.24 (one hole, 1 in. in diam., in each angle)
10.93 sq. ins., net.

Stringers for 10-ft. Panel.

Dead-load = 450 lbs. per lin. ft. Live-load: two driving-wheels weighing 30,000 lbs. each, spaced 7-ft. centers. The

maximum live-load moment will occur with a wheel at the center of span.

SHEAR :

$$\text{Dead-load} = 450 \times 5 = 2{,}250$$

$$\text{Live-load} = 30{,}000 \times \frac{13}{10} = 39{,}000$$

$$\text{Impact} = \frac{39{,}000^2}{39{,}000 + 2{,}250} = 36{,}900$$

78,150 lbs.

A 36 x ⅜-in. web-plate will be used, as before.

MOMENT:

$$\text{Dead-load} = \frac{450 \times 10^2}{8} = 5{,}600$$

$$\text{Live-load} = \frac{30{,}000 \times 10}{4} = 75{,}000$$

$$\text{Impact} = \frac{75{,}000^2}{75{,}000 + 5{,}600} = 69{,}800$$

150,400 ft.-lbs.

Effective depth of stringer = 2.85 ft.

Flange-stress = 150,400 ÷ 2.85 = 52,800 lbs.

Flange-area required = 52,800 ÷ 16,000 = 3.30 sq. ins.

Equivalent area provided: ⅛ of 36 x ⅜-in. web-plate = 1.69
2 angles 3½ x 3½ x ⅜-in. = 4.21
(one hole 1 in. in diam., in each angle)
sq. ins. net. 5.90

Intermediate Floorbeams.

Effective length of floorbeam = 17.5 ft.

The stringer-concentrations are 8 ft. apart and 4.75 ft. from center of trusses.

The weight of floorbeam is assumed to be 3,000 lbs., which is a distributed load.

Dead-load concentrations from stringers = 450×20 = 9,000 lbs.

Live-load concentrations from stringers, as found previously for truss members $B\ b$ and $D\ d$, = 74,700 lbs.

END SHEAR:

Dead-load	$=\dfrac{3{,}000}{2}+9{,}000$	=	10,500
Live-load		=	74,700
Impact	$=\dfrac{74{,}700^2}{74{,}700+10{,}500}$	=	65,500
			150,700 lbs.

Area required in web-plate = 150,700 ÷ 10,000 = 15.07 sq. ins.

A 42 x ⅜-in. web-plate = 15.75 sq. ins., will be used.

MOMENT:

Dead-load	$=\dfrac{3{,}000\times 17.5}{8}+(9{,}000\times 4.75)$ =	49,300
Live-load	$= 74{,}700\times 4.75$ =	354,800
Impact	$\dfrac{354{,}800^2}{354{,}800+49{,}300}$ =	311,800
		715,900 ft.-lbs.

Effective depth of floorbeam = 3.25 ft.
Flange-stress = 715,900 ÷ 3.25 = 220,300 lbs.
Flange-area required = 220,300 ÷ 16,000 = 13.77 sq. ins.
Section used: ⅛ of 42 x ⅜-in. web-plate = 1.97
2 angles 6 x 6 x ⅝-in. = 11.72 (2 holes, 1 in. in diam., in each angle).
13.69 sq. ins., net

END FLOORBEAMS.

The effective length, position of stringer-concentrations and weight of floorbeam, as before.

The dead-load concentrations from stringers = 450 × 10 = 4,500 lbs.

The live-load concentrations are equal to the maximum end shear for stringers = 57,500 lbs.

END SHEAR:

Dead-load	$= \dfrac{3{,}000}{2} + 4{,}500 =$	6,000
Live-load	$=$	57,500
Impact	$= \dfrac{57{,}500^2}{57{,}500 + 6{,}000} =$	52,000
		115,500 lbs.

A 42 x ⅜-in. web-plate will be used as before.

MOMENT:

Dead-load	$= \dfrac{3{,}000 \times 17.5}{8} + (4{,}500 \times 4.75) =$	27,900
Live-load	$= 57{,}500 \times 4.75$	$=$ 273,100
Impact	$= \dfrac{273{,}100^2}{273{,}100 + 27{,}900}$	$=$ 247,800
		548,800 ft.-lbs.

Effective depth of floorbeam = 3.25 ft.
Flange-stress = 548,800 ÷ 3.25 = 168,900 lbs.
Flange-area required = 168,900 ÷ 16,000 = 10.55 sq. ins.

Section used: ⅛ of 42 x ⅜-in. web-plate	= 1.97
2 angles 6 x 6 x ½-in.	= 9.50 (2 holes, 1 in. in diam., in each angle).
	11.47 sq ins., net.

Loading Girders.

The trusses rest on the extremities of loading girders A, which latter are supported on the center of loading girders B. Loading girders B are framed into the radial girders C, which in turn distribute the loads between the circular girder and the center pivot, as shown in Fig. 17.

Loading Girders A.

Points of support, 10 ft. c. to c.
Stringer-concentrations, 8 ft. c. to c.
Truss-concentrations, 17.5 ft. c. to c.
Assumed weight of girder, 300 lbs. per lin. ft.
Live-load, 6,500 lbs. per lin. ft. of bridge.

Stringer-concentrations: Dead-load $= 450 \times \frac{20+10}{2} = 6,750$ lbs.

Live-load $= \frac{6,500}{2} \times \frac{20+10}{2} = 48,700$ lbs.

The dead-load at panel-points a and e of truss $= 14,250$ lbs.

The dead-load at panel-points b, c and d of truss $= 19,000$ lbs.

The dead-load concentrations at ends of girder are equal to the loads at a, b, c, d, and e, less the stringer-concentration on girder, $= (14,250 \times 2) + (19,000 \times 3) - 6,750 = 78,750$ lbs.

The live-load concentrations at ends of girder are equal to the loads at b, c and d, less the reaction at a due to these loads. $= (65,000 \times 3) - 84,700 = 110,300$ lbs.

The maximum shear in loading girder A is just outside the points of support. The dead-load shear is equal to the concentration at end, plus the weight of girder between point of support and end.

The live-load shear is equal to the concentration at end.

SHEAR:

Dead-load	$= 78,750 + (300 \times 4.5)$	$= 80,100$
Live-load		$= 110,300$
Impact	$= \frac{110,300^2}{110,300+80,100}$	$= 63,900$
		254,300 lbs.

Area required in web-plate $= 254,300 \div 10,000 = 25.43$ sq. ins.

A 42 x $\frac{5}{8}$-in. web-plate $= 26.25$ sq. ins., will be used.

The maximum moment is at points of support.

MOMENT:

Dead-load	$= (78,750 \times 3.75) + \left(\frac{300 \times 4.5^2}{2}\right)$	$= 298,300$
Live-load	$= 110,300 \times 3.75$	$= 413,600$
Impact	$= \frac{413,600^2}{413,600+298,300}$	$= 240,300$
		952,200 ft.-lbs.

The effective depth, or distance c. to c. of gravity of flanges = 3.37 ft.

Flange-stress = 952,200 ÷ 3.37 = 282,500 lbs.

Flange-area required = 282,500 ÷ 16,000 = 17.65 sq. ins.

Section provided:

⅛ of 42 x ⅝-in. web-plate	=	3.28
2 angles 6 x 6 x ½-in.	=	9.50 (2 holes 1 in. in diam. in each angle).
1 plate 13 x ½-in.	=	5.50 (2 holes 1 in. in diam.)
		18.28 sq. ins., net.

Loading Girders *B*.

Span of girder = 5.86 ft.; loaded at center.

DEAD-LOAD:

Loading girder *A* = 300 lbs. × 9.50 ft.	= 2,850
Stringer-concentration	= 6,750
Truss-concentration	= 78,750
	88,350 lbs.

LIVE-LOAD:

Stringer-concentration	= 48,700
Truss-concentration	= 110,300
	159,000 lbs.

SHEAR:

Dead-load $= \frac{88,350}{2} = 44,200$

Live-load $= \frac{159,000}{2} = 79,500$

Impact $= \frac{79,500^2}{79,500 + 44,200} = 51,100$

174,800 lbs.

Area required in web-plate = 174,800 ÷ 10,000 = 17.48 sq. ins.

A 27 x ¾-in. web-plate (= 20.25 sq. ins.,) will be used.

MOMENT:

Dead-load	$= \dfrac{88{,}350 \times 5.86}{4} =$	129,400
Live-load	$= \dfrac{159{,}000 \times 5.86}{4} =$	233,000
Impact	$= \dfrac{233{,}000^2}{233{,}000 + 129{,}400} =$	149,800
		512,200 ft.-lbs.

Effective depth of girder = 2 ft.
Flange-stress = 512,200 ÷ 2 = 256,100 lbs.
Flange-area required = 256,100 ÷ 16,000 = 16 sq. ins.
Section provided:

⅛ of 27 x ¾-in. web-plate	= 2.53	
2 angles 6 x 6 x ¾-in.	= 13.88	(2 holes, 1 in. in diam., in each angle).
	16.41 sq. ins., net.	

Loading Girders *C*.

The span = 9 ft. Load is applied 1.35 ft. from circular girder, and 7.65 ft. from center pivot.

The total load, including impact, is equal to the end shear or reaction of girder *B*, = 174,800 lbs.

Shear between point of loading and circular girder $= 174{,}800 \times \dfrac{7.65}{9} = 148{,}600$ lbs.

Area required in web-plate = 148,600 ÷ 10,000 = 14.86 sq. ins.
A 27 x ⅝-in. web-plate, = 16.87 sq. ins., will be used.
Moment at point of loading = 148,600 × 1.35 = 200,600 ft.-lbs.
Effective depth of girder = 1.95 ft.
Flange-stress = 200,600 ÷ 1.95 = 102,900 lbs.
Flange-area required = 102,900 ÷ 16,000 = 6.43 sq. ins.
Section provided:

⅛ of 27 x ⅝-in. web plate	= 2.11	
2 angles 6 x 3½ x ⅜-in.	= 3.54	(2 holes, 1 in. in diam., in each angle)
	5.65 sq. ins., net	

The 6-in. legs of flange-angles will be vertical, so that the required number of rivets between point of loading and circular girder may be provided.

Circular Girder.

The diameter of circular girder = 18 ft., and the circumference = 18π = 56.6 ft. The distance between concentrations from radial girders $C = \frac{56.6}{8} = 7.07$ ft. The circular girder may be considered as though it were continuous over supports 7.07 ft. apart, and loaded uniformly between supports with a load equal to the reaction of one radial girder. The points of maximum moment will be at the radial-girder concentrations, and the moment at these points will be assumed to be equal to three-fourths of the bending moment for a simple span of 7.07 ft. The maximum shear will be equal to one-half of the radial-girder concentration.

$$\text{Shear} = \frac{148,600}{2} = 74,300 \text{ lbs.}$$

Area required in web-plate = $74,300 \div 10,000$ = 7.43 sq. ins.
A 27 x $\frac{3}{8}$-in. web-plate, = 10.12 sq. ins., will be used.

$$\text{Moment} = \frac{148,600 \times 7.07}{8} \times \frac{3}{4} = 98,500 \text{ ft.-lbs.}$$

The effective depth of girder = 1.95 ft.
Flange-stress = $98,500 \div 1.95$ = 50,500 lbs.
Flange-area required = $50,500 \div 16,000$ = 3.15 sq. ins.

It will be necessary to make the flanges considerably heavier than the above, as double rows of rivets will be required in the vertical legs of flange-angles, and some allowance should be made for horizontal bending moments due to curvature of girder. Two 6 x $3\frac{1}{2}$ x $\frac{3}{8}$-in. angles will be used for the top flange, and two 6 x $3\frac{1}{2}$ x $\frac{5}{8}$-in. angles for the bottom flange. The 6-in. legs will in both cases be vertical. The bottom angles are made extra heavy to provide a more efficient support for the upper tread, and to allow for planing on the bottom to insure a perfect bearing on this tread.

In computing the rivet-spacing in the bottom flange-angles, the reactions of the rollers must be combined with the longi-

tudinal shear. The case is similar to that of a deck plate-girder, but turned upside down. The required pitch for ⅞-in. rivets will be computed as follows:

Total vertical shear = 74,300 lbs.

Vertical load on rivets per lin. in. $= \dfrac{148{,}600}{7.07 \times 12} = 1{,}750$ lbs.

Vertical distance c. to c. of gravity of flanges = 23.4 ins.

Value of one ⅞-in. rivet bearing on ⅜-in. plate = 7,220 lbs.

Longitudinal shear per lin. in. at center of gravity of flanges = $\dfrac{74{,}300}{23.4} = 3{,}175$ lbs.

Assuming for equivalent flange-area

⅛ of 27 x ⅜-in. web-plate = 1.25

2 angles 6 x 3½ x ⅜-in. = 5.34 (2 holes, 1 in. in diam., in each angle).

6.59 sq. ins., net.,

then, longitudinal shear per lin. in. on rivets $= 3{,}175 \times \dfrac{5.34}{6.59} =$ 2,570 lbs.

Total stress on rivets of bottom flange per lin. in. = $\sqrt{2{,}570^2 + 1{,}750^2} = 3{,}110$ lbs.

Required spacing of rivets = $7{,}220 \div 3{,}110 = 2.32$ ins.

For uniformity, the rivet pitch in both top and bottom flanges should be about 2¼ ins., staggered, throughout

Rollers and Center Pivot.

When the bridge is being swung, only the dead-load requires to be considered, which is as follows:

Dead-load above turntable	$= (14{,}250 \times 8) + (19{,}000 \times 12)$	= 342,000
Weight of loading girders—assumed		= 40,000
Total dead-load on center pier		= 382,000 lbs.

The dead-load on rollers $= 382{,}000 \times \dfrac{7.65}{9} = 324{,}700$ lbs.

The dead-load on center pivot $= 382{,}000 \times \dfrac{1.35}{9} = 57{,}300$ lbs.

Assuming rollers 10 ins. in diameter, the permissible load per lin. in. = $1,200\sqrt{10}$ = 3,790 lbs.; and the total bearing-length of rollers required = 324,700 ÷ 3,790 = 86 ins. There will be 24 rollers used, of 10-in. diameter and 5-in. face, giving a total bearing of 24 × 5 = 120 lin. ins.

The upper treads will be of rolled steel, 6½ ins. wide and 1¾ ins. thick on center line. The lower treads will be of cast iron, 4½ ins. high on center line.

The load per square inch on center pivot, when bridge is swinging, should not exceed 2,000 lbs. Then, bearing area required = 57,300 ÷ 2,000 = 29 sq. ins. The diameter of the pivot to be used = 7 ins., and its area = 38 sq. ins. There will be three disks under pivot; the center one of phosphor-bronze, 1½ ins. thick at center, convex both top and bottom; the upper and lower disks of cast steel, 1 in. thick at center and concave on one side to fit center disk.

The base of the center casting should be designed for the dead- and live-loads with impact. The maximum live-load on center pier occurs when the bridge is loaded over all, and is equal to the total live-load, including loads at panel-points *e* and *f*, less the end reactions for this condition of loading, as already determined. The regular panel loads = 53,000 lbs.; the loads at panel-points *e* and *f* = 53,000 × ¾ = 39,750 lbs.; and the end reactions = 58,600 lbs.

The total live-load on center pier = (53,000 × 12) + (39,750 × 4) − (58,600 × 4) = 560,600 lbs.; and the live load on center pivot

$$= 560,600 \times \frac{1.35}{9} = 84,100 \text{ lbs.}$$

Maximum load on center casting:

Dead-load	= 57,300
Live-load	= 84,100
Impact	= 50,300
	191,700 lbs.

Taking the allowable bearing on masonry at 400 lbs. per sq. in., the required area of base = 191,700 ÷ 400 = 479 sq. ins. The diameter of the base used = 26 ins., and its area = 530 sq. ins.

Bottom Laterals.

The horizontal truss in the plane of the bottom chords should be treated as a partially continuous girder in the same manner as the vertical trusses, using the same coefficients for reactions. Length of panels, 20 ft.; depth, 17.5 ft.; length of diagonals = $\sqrt{20^2+17.5^2} = 26.57$ ft.

Panel dead-load, 150 lbs. × 20 ft. = 3,000 lbs.; panel live-load, 400 lbs. × 20 ft. = 8,000 lbs.

The dead-load concentration at panel-point a = 1,500 lbs.

Stresses—Case I.

1st diagonal $= 1{,}500 \times \frac{26.57}{17.5}$ $= -2{,}300$

2d " $= (1{,}500+3{,}000) \times \frac{26.57}{17.5}$ $= -6{,}800$

3d " $= \left(1{,}500+(3{,}000\times 2)\right) \times \frac{26.57}{17.5}$ $= -11{,}400$

4th " $= \left(1{,}500+(3{,}000\times 3)\right) \times \frac{26.57}{17.5}$ $= -15{,}300$

For Case II the reaction $R_1 = 3{,}000 \times 1.106 = 3{,}300$ lbs.; and the stress in 1st diagonal $= 3{,}300 \times \frac{26.57}{17.5} = -4{,}800$ lbs. The stresses in the other diagonals will be much smaller than those already found for Case I, so it will be unnecessary to figure them for this condition.

Reactions for Case III.

Loads at b, c, and d, giving maximum stress in 1st diagonal: $R_1 = 8{,}000 \times 1.303 = 10{,}400$ lbs.

Load at b only, giving maximum stress in 2d diagonal: $R_1 = 8{,}000 \times 0.701 = 5{,}600$ lbs.

Load over all, giving maximum stresses in 3d and 4th diagonals: $R_1 = 8{,}000 \times 1.106 = 8{,}800$ lbs.

Stresses—Case III.

1st diagonal $= 10{,}400 \times \frac{26.57}{17.5}$ $= -15{,}800$

2d " $= (5{,}600-8{,}000) \times \frac{26.57}{17.5}$ $= -3{,}600$

3d diagonal $= \left(8{,}800-(8{,}000\times 2)\right) \times \dfrac{26.57}{17.5} \qquad = -10{,}900$

4th " $= \left(8{,}800-(8{,}000\times 3)\right) \times \dfrac{26.57}{17.5} \qquad = -23{,}100$

The maximum stresses are shown on stress diagram, Fig. 17. For the 1st diagonal Case II is combined with Case III; but, for the other diagonals, Case I is combined with Case III. The impact formula is not applied to wind stresses.

The top-lateral stresses, which are very small, will not be investigated.

END-LIFTS.

A load on one arm only will tend to lift the outer end of the other arm; therefore it must either be latched down, or lifted sufficiently, to prevent hammering. As it is difficult to hold down the ends of a swing-span effectually, the latter method will be employed.

The reactions R_4 for a load of 1 lb. at b, c and d have already been computed. They are as follows:

R_4 for load at b	$= -0.049$
c	$= -0.079$
d	$= -0.069$
	-0.197

The panel-concentration when one arm only is loaded $= 65{,}000$ lbs.

Then the maximum negative reaction $= 65{,}000 \times (-0.197) = -12{,}800$ lbs.

Therefore the ends must be lifted sufficiently to give a positive dead-load reaction, when there is no live-load on bridge, somewhat greater than 12,800 lbs. This reaction will be assumed at 15,000 lbs., to which 60% will be added for friction of screws and gears, making 24,000 lbs.

A vertical screw will be used under each corner of the bridge, connected by bevel-gears with a horizontal cross-shaft under each end floorbeam. A longitudinal shaft near the center of bridge will be connected by miter-gears with the end cross-shafts, and by bevel-gears with a vertical shaft near the center to which latter the operating lever will be applied. The vertical screws

will be 4 ins. in diam., with $\frac{1}{4}$-in. double thread, equal to $\frac{1}{2}$-in. pitch. The depth of thread will be about $\frac{1}{4}$-in.; therefore the diameter of screw, on center line of thread, will be 3.75 ins.; and the circumference $(3.75 \times 3.14) = 11.78$ ins. Then the force required at center line of thread $= 24{,}000 \times \frac{0.5}{11.78} = 1{,}020$ lbs. The bevel-gears used on the vertical screws will be $19\frac{1}{16}$ ins. in pitch diameter, with 40 teeth of $1\frac{1}{2}$-in. pitch and 3-in. face; and the force required at pitch-line of these gears is equal to that required at the screws, multiplied by the radius of screws and divided by the radius of gears, $= 1{,}020 \times \frac{1.87}{9.5} = 200$ lbs.

The bevel-gears used at the ends of the cross-shafts to connect with the vertical screws will be $8\frac{9}{16}$ ins. in pitch-diameter, with 18 teeth of $1\frac{1}{2}$-in. pitch and 3-in. face. The miter-gears connecting the cross-shafts with the longitudinal shaft will be $9\frac{1}{2}$ ins. in pitch-diameter, with 20 teeth of $1\frac{1}{2}$ in. pitch and 3-in. face. These miter-gears will not change the force, as they are all of the same diameter. At the connection of the longitudinal shaft with the vertical shaft near center of bridge, there will be a bevel-wheel, $19\frac{1}{16}$ ins. in pitch-diameter with 40 teeth of $1\frac{1}{2}$-in. pitch and 3-in. face, keyed to the longitudinal shaft; and a bevel-pinion, $8\frac{9}{16}$ ins. in pitch-diameter, with 18 teeth of $1\frac{1}{2}$-in. pitch and 3-in. face, keyed to the vertical shaft. The force required at the pitch-line of these gears is equal to the loads on the four gears at ends of cross-shafts, multiplied by their common radius and divided by the radius of the $19\frac{1}{16}$-in. bevel wheel on longitudinal shaft, $= 200 \times 4 \times \frac{4.25}{9.5} = 360$ lbs. The operating lever will have a working radius of 60 ins. and the force required at its end is equal to the load at the pitch-line of the $8\frac{9}{16}$-in. pinion on vertical shaft, multiplied by its radius and divided by radius of lever $= 360 \times \frac{4.25}{60} = 25$ lbs.

It has been found that the average bridge-tender will push about 50 lbs. on a turning lever while walking at the rate of from 150 ft. to 200 ft. per min. Thus one man will easily operate the end-lifts.

Since the pitch of thread on end-screws = $\frac{1}{4}$-in., one revolution will lift the ends $\frac{1}{4}$-in.; and the corresponding number of revolutions of the operating lever will be $1\times\frac{40}{18}\times\frac{40}{18}=5$. The man will walk 10 ft.$\times 3.14\times 5$ = 157 ft. Thus it will require somewhat less than one minute to raise the ends of bridge $\frac{1}{4}$-in.

Shafting.

A bridge-shaft is subject to both twisting and bending moments.

These may be reduced to an equivalent twisting moment, for which it can be proportioned. The following formula, from Unwin, will be used in this investigation:

$$T_1 = M+\sqrt{M^2+T^2},$$

in which M = bending moment in in.-lbs.
T = twisting moment in in.-lbs.
T_1 = equivalent twisting moment in in.-lbs. for combination of M and T.

The section modulus for twisting of a solid round shaft $=\frac{\pi d^3}{16}$, which is just twice that for bending. A fiber stress of 9,000 lbs. per sq. in. will be allowed for shearing; then, having found the equivalent twisting moment, the size of shaft required may be taken from a table of *bending* moments on pins, figured for 18,000 lbs. fiber-stress.

At the connection of the longitudinal shaft with the vertical shaft near center of bridge, the load at pitch-line of gears = 360 lbs.; the distance from pitch-line to center of bearing for longitudinal shaft will be about 5 ins.; and the radius of gear = 9.5 ins. Then $M = 360\times 5 = 1{,}800$ in.-lbs., $T = 360\times 9.5 = 3{,}400$ in.-lbs., and $T_1 = 1{,}800+\sqrt{1{,}800^2+3{,}400^2} = 5{,}650$ in.-lbs. A shaft $1\frac{3}{4}$ ins. in diameter, the twisting value of which = 5,960 in.-lbs. would answer, but 2-in. shafting will be used both for the longitudinal and the end cross-shafts, as it is considered unadvisable to use anything smaller in bridgework.

The vertical shaft will be $2\frac{1}{4}$ ins. in diameter, in order that it may have a 2-in. square at the top for the connection of the operating lever.

Strength of Gear-Teeth.

The following formulas, from Kent, have been found to give satisfactory results.

For spur-gears, $W = s\,p\,f\,y.;$ (1)

For bevel-gears, $W = s\,p\,f\,y\frac{d}{D};$ (2)

in which W = load the tooth will transmit.

s = safe working stress = 8,000 lbs. for iron castings and 20,000 lbs. for steel castings, when speed does not exceed 100 ft. per min.

p = pitch.

f = face.

y = a factor depending on form of tooth, the value of which is given in the table below.

D = large diameter of bevel-gear.

d = small diameter of bevel-gear.

Values of y for Different Forms of Gear Teeth.

Number of Teeth	Factor for strength y			Number of Teeth	Factor for strength y		
	20° Involute	15° Involute and Cycloidal	Radial Flanks		20° Involute	15° Involute and Cycloidal	Radial Flanks
12	.078	.067	.052	27	.111	.100	.064
13	.083	.070	.053	30	.114	.102	.065
14	.088	.072	.054	34	.118	.104	.066
15	.092	.075	.055	38	.122	.107	.067
16	.094	.077	.056	43	.126	.110	.068
17	.096	.080	.057	50	.130	.112	.069
18	.098	.083	.058	60	.134	.114	.070
19	.100	.087	.059	75	.138	.116	.071
20	.102	.090	.060	100	.142	.118	.072
21	.104	.092	.061	150	.146	.120	.073
23	.106	.094	.062	300	.150	.122	.074
25	.108	.097	.063	rack	.154	.124	.075

Using the value of y for 15° involute and cycloidal teeth, the strength of the bevel-pinion, on vertical shaft at connection with the longitudinal shaft, will now be computed. The load on tooth = 360 lbs.; pitch-diam. = $8\frac{5}{16}$ ins.; pitch = $1\frac{1}{2}$ ins.; face = 3 ins.; number of teeth = 18; small diam. = 6 ins. (about); y = 0.083.

hen $W = s\,p\,f\,y\frac{d}{D} = 8{,}000 \times 1.5 \times 3 \times 0.083 \times \frac{6}{8.5} = 2{,}100$ lbs.

Thus the teeth of this pinion are much stronger than required for the load, but there would be little saving in using a smaller pitch on ordinary cast-iron gears.

It will be unnecessary to figure the capacity of the other end-lift gears, as the loads on their teeth are all smaller than that on the pinion just considered.

TURNING MACHINERY.

The following forces are to be overcome:

(1) The inertia of the bridge.
(2) The friction of rollers.
(3) The friction due to load on center pivot.
(4) The friction on side of pivot due to wind force.
(5) The friction of gearing.

These forces will be reduced to an equivalent resistance at the rack-circle, where the power to overcome it will be applied. The data for computing the various resistances to turning are as follows:

Length of bridge, 170 ft.
Width of bridge, 17.5 ft.
Radius of rack-circle, 9 ft. $10\frac{1}{16}$ ins. = 9.83 ft.
Total weight of bridge, 382,000 lbs.
Assumed time for opening, 3 minutes = 180 seconds.

The coefficient for rolling friction will be taken at 0.005; ar for sliding friction, at 0.100.

(1) Force Required at Rack-Circle to Overcome Inertia Bridge:

The polar moment of inertia of the bridge about center pi is given, with sufficient accuracy, by the formula

$$I = W\frac{l^2 + w^2}{12},$$

in which I = polar moment of inertia.
W = total weight of bridge.
l = length of bridge.
w = width of bridge.

Then $I = 382{,}000 \times \frac{170^2 + 17.5^2}{12} = 929{,}700{,}000.$

An equivalent weight, concentrated at rack-circle, which would have this same moment of inertia about center pivot $= \frac{I}{R^2}$, in which R = radius of rack-circle.

Equivalent weight at rack-circle $= \frac{929,700,000}{9.83^2} = 9,620,000$ lbs.

The circumference of rack-circle $= 9.83 \times 2 \times 3.14 = 62$ ft., and one-quarter of the circumference = 15.5 ft. Since the bridge is required to make one-quarter of a revolution in 180 seconds, the average velocity at rack-circle $= \frac{15.5}{180} = 0.086$ ft. per sec. The maximum velocity, which is reached at the end of 90 seconds $= 0.086 \times 2 = 0.172$ ft. per sec.; and the acceleration, or rate of increase of velocity, $= \frac{0.172}{90} = 0.0019$ ft. per sec.

The acceleration of a body is proportional to the applied force. Then, since the acceleration of a body due to the force of gravity is known, the acceleration of the same body due to any other force acting on it will be in the ratio of this applied force to the force of gravity, or weight. In other words, applied force : weight : : acceleration due to applied force: acceleration of gravity.

Therefore, the force required to accelerate a body a given amount is equal to the weight of the body multiplied by this acceleration and divided by the acceleration of gravity; or, stated by formula,

$$F = \frac{W a}{g},$$

in which W = the force of gravity acting on the body.
F = any other force acting on the same body.
g = acceleration due to force of gravity = 32.2 ft. per sec.
a = acceleration due to force F.

Then the force required at rack-circle to accelerate the equivalent weight of 9,620,000 lbs. at the rate of 0.0019 ft. per sec. $= \frac{9,620,000 \times 0.0019}{32.2} = 567$ lbs.

(2) Force Required at Rack-Circle to Overcome Friction of Rollers:

Load on rollers = 324,700 lbs. Coefficient of friction = 0.005. Radius of wheel-circle = 9 ft. Radius of rack-circle = 9.83 ft. Then:

$$324{,}700 \times 0.005 \times \frac{9}{9.83} = 1{,}486 \text{ lbs.}$$

(3) Force Required at Rack-Circle to Overcome Friction Due to Load on Center Pivot:

Load on pivot = 57,300 lbs. Coefficient of friction = 0.1. Radius of pivot = $3\frac{1}{2}$ ins. = 0.3 ft. Center of frictional resistance is $\frac{2}{3}$ of radius from center, = 0.2 ft. Then

$$57{,}300 \times 0.1 \times \frac{0.2}{9.83} = 116 \text{ lbs.}$$

(4) Force Required at Rack-Circle to Overcome Friction on Side of Pivot Due to Wind:

Total wind pressure on bridge = 170 (ft.) × 300 lbs. = 51,000 lbs. Coefficient of friction = 0.1.

Frictional resistance is at circumference of pivot = 0.3 ft. from center. Then:

$$51{,}000 \times 0.1 \times \frac{0.3}{9.83} = 155 \text{ lbs.}$$

The total force required at rack-circle is as follows:

(1)	= 567
(2)	= 1,486
(3)	= 116
(4)	= 155
	2,324 lbs.

To allow for friction of gearing, 25% will be added to this total, thus: 2,324 + 581 = 2,905 lbs., which is the load on rack-pinion.

The arrangement of turning machinery is shown in Fig. 19. The rack will be 19 ft. $8\frac{1}{8}$ ins. in pitch diameter, of $2\frac{11}{16}$-in. pitch and 6-in. face. The rack-pinion will be $12\frac{13}{16}$ ins. in pitch-diameter, with 15 teeth of $2\frac{11}{16}$-in. pitch and $6\frac{1}{2}$-in. face. The permissible load on a tooth of this pinion $= 8{,}000 \times 2\frac{11}{16} \times 6\frac{1}{2} \times 0.075 = 10{,}480$ lbs. It will be keyed to the lower end of a vertical

shaft; and at the upper end of this shaft will be keyed a spur-wheel, 22$\frac{29}{32}$ ins. in pitch diameter, with 48 teeth of 1$\frac{1}{2}$-in. pitch and 3-in. face. This spur-wheel will gear with a pinion, 5$\frac{3}{4}$ ins. in pitch-diameter, with 12 teeth of 1$\frac{1}{2}$-in. pitch and 3$\frac{1}{4}$-in. face, keyed to another vertical shaft, to which the turning lever will be attached. The force required at the pitch-line of this pair of gears is equal to the total force required at rack-circle (including friction) multiplied by the radius of rack-pinion and divided by the radius of spur-wheel $= 2{,}905 \times \frac{6.4}{11.44} = 1{,}625$ lbs. The permissible load on a tooth of the 5$\frac{3}{4}$-in. pinion $= 8{,}000 \times 1\frac{1}{2} \times 3\frac{1}{4} \times 0.067 = 2{,}613$ lbs. The force required at end of turning lever is equal to the force required at pitch-line of

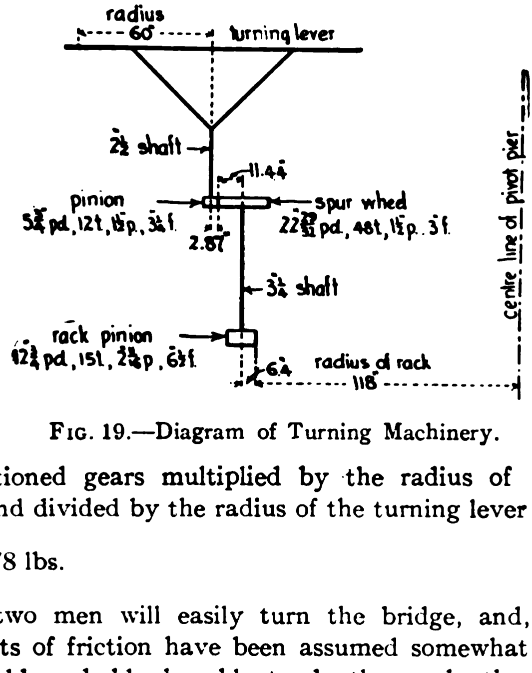


FIG. 19.—Diagram of Turning Machinery.

last-mentioned gears multiplied by the radius of the 5$\frac{3}{4}$-in. pinion and divided by the radius of the turning lever $= 1{,}625 \times \frac{2.87}{60} = 78$ lbs.

Thus two men will easily turn the bridge, and, since the coefficients of friction have been assumed somewhat large, one man would probably be able to do the work, though not so quickly.

The number of revolutions of turning lever to one-quarter revolution of bridge $= \frac{1}{4} \times \frac{118}{6.4} \times \frac{11.44}{2.87} = 18.37$; and the distance

that the men will walk $= 18.37 \times 3.14 \times 10$ ft. $= 576$ ft. Then, if they walk at the rate of 200 ft. per min., it will require $\frac{576}{200} =$ 2.88 minutes to open or close bridge.

Rack-Pinion Shaft.—The distance from center of rack pinion to center of bearing will be about $7\frac{1}{2}$ ins; radius of pitch-line = 6.4 ins.; load on tooth of pinion = 2,905 lbs. Then $M = 2{,}905 \times 7.5 = 21{,}800$ in.-lbs., and $T = 2{,}905 \times 6.4 = 18{,}600$ in.-lbs. $T_1 = M + \sqrt{M^2 + T^2} = 21{,}800 + \sqrt{21{,}800^2 + 18{,}600^2} = 50{,}500$ in.-lbs.

A $3\frac{1}{4}$-in. shaft will be used, the twisting value of which = 60,700 in.-lbs.

Turning-Shaft.—The shaft to which turning lever will be applied will be $2\frac{1}{2}$ ins. in diameter, with a 2-in. square at the top.

Deflections and Camber.

The deflection of a framed structure is due to the shortening of all the compression members and the lengthening of all the tension members, and may be obtained at any panel-point by the formula:

$$\Delta = \Sigma \frac{p\,u\,l}{E}.*$$

In which Δ = deflection in inches at the required point.
Σ = the sign of summation.
p = stress per sq. in. of gross area in each member of the truss.
u = stress in each member due to a load of 1 lb. placed at the point where deflection is required.
l = length of each member in inches.
E = modulus or coefficient of elasticity = 29,000,000 lbs. for mild steel.

The factor $\frac{pl}{E}$ represents the change in length of a member due to compression or tension; and this change in length, multiplied by the factor u for the member, will give the amount of deflection (at the point where the 1-lb. load is applied) due to this change; and the total deflection at this point will be equal to the sum of the deflections due to the change in length of each

* See Johnson's "Theory and Practice of Modern Framed Structures."

member of the truss. If a compression member be shortened, or a tension member lengthened, by any other means, then this change of length multiplied by the factor u for the member will give the deflection as before. If a compression member be lengthened, or a tension member be shortened, the change of length multiplied by the factor u will give the amount that the point will be raised.

In the present example, it is required to find the deflection at the ends of bridge when the ends are unsupported. Each arm is then a cantilever, and it will only be necessary to consider the distortions of the truss-members of one arm. The load of 1 lb. will be placed at the end a, and the stresses, or coefficients u, figured for the various members, as follows:

COEFFICIENTS u.

$a\,B$ & $c\,D$	$= 1 \times \frac{34.41}{28} = -1.230$
$B\,c$ & $D\,e$	$= 1 \times \frac{34.41}{28} = +1.230$
$a\,b$ & $b\,c$	$= 1 \times \frac{20}{28} = +0.714$
$B\,C$ & $C\,D$	$= 1 \times \frac{40}{28} = -1.428$
$c\,d$ & $d\,e$	$= 1 \times \frac{60}{28} = +2.142$
$D\,E$ & $E\,F$	$= 1 \times \frac{80}{28} = -2.856$
$e\,f$	$= 1 \times \frac{80}{28} = +2.856$

A table will now be constructed, using the stresses, Case I, and gross areas of members, as shown on stress-diagram. In making the calculations for this table, it should be remembered that a minus multiplied by a minus yields a plus. Members $E\,F$ and $e\,f$ will be taken at one-half their length only, as the remaining half-lengths belong to the other arm of bridge. Since a load at a induces no stresses in any of the vertical members of truss, there are no coefficients u for them, and any distortions

of these members will have no effect on the end-deflection. For convenience, the division by E will be made after summing up the various values for $p\,u\,l$.

TABLE FOR DEFLECTION AT *a* DUE TO DEAD-LOAD.

Member	Stress	Area	p	u	l	$p\,u\,l$
$a\,B$	− 17.500	17.64	− 1.000	− 1.230	413	+ 509.000
$B\,c$	+ 40.800	14.70	+ 2.780	+ 1.230	413	+ 1.400.000
$c\,D$	− 64.300	15.88	− 4.050	− 1.230	413	+ 2.060.000
$D\,e$	+ 87.500	26.48	+ 3.300	+ 1.230	413	+ 1,680.000
$a\,b$	+ 10.300	14.70	+ 700	+ 0.714	240	+ 120.000
$b\,c$	+ 10.300	14.70	+ 700	+ 0.714	240	+ 120.000
$c\,d$	+ 71.500	14.70	+ 4.860	+ 2.142	240	+ 2.500.000
$d\,e$	+ 71,500	14.70	+ 4.860	+ 2.142	240	+ 2,500,000
$e\,f$	+ 122.400	14.70	+ 8,350	+ 2.856	$\frac{120}{2}$	+ 1,430.000
$B\,C$	− 34.000	14.70	− 2,310	− 1.428	240	+ 790.000
$C\,D$	− 34.000	14.70	− 2.310	− 1.428	240	+ 790.000
$D\,E$	− 122,400	17.64	− 6.950	− 2.856	240	+ 4,760.000
$E\,F$	− 122,400	17.64	− 6.950	− 2.856	$\frac{120}{2}$	+ 1.190.000
						+ 19.849.000

$$\text{Then, deflection at } a = \Sigma \frac{p\,u\,l}{E} = \frac{19{,}849{,}000}{29{,}000{,}000} = 0.69 \text{ in.}$$

To counteract the deflection due to live-load, the bridge will be cambered at c and h by adding ¼-in. to each of the members $B\,C$, $C\,D$, $G\,H$, and $H\,I$. This increase in length of top chords will cause additional deflections at a and j, equal to the coefficient u for members $B\,C$ and $C\,D$, multiplied by the increase in length of these members. Then, the deflection at a caused by increasing the length of members $B\,C$ and $C\,D = 1.428 \times$ ¼-in. $\times\ 2 = 0.71$ in. The total deflection at a will now be $0.69 + 0.71 = 1.4$ ins. Part of this deflection will be counteracted by the end-screws, and the balance by shortening the members $D\,E$ and $F\,G$.

It will now be necessary to determine how much the ends require to be lifted in order that the reactions will be equal to the assumed loads of 15,000 lbs. on the screws. This will be done by placing a load of 15,000 lbs. at a, and computing the end-deflection due to this load. The amount necessary to raise the end to give a reaction of 15,000 lbs. will evidently be equal to the deflection at this point caused by a load placed there,

which is equal to this reaction. The stresses caused by a load of 15,000 lbs. at a are as follows:

STRESSES FOR LOAD OF 15,000 LBS. AT a.

$$aB \,\&\, cD = 15{,}000 \times \frac{34.41}{28} = -18{,}500$$

$$Bc \,\&\, De = 15{,}000 \times \frac{34.41}{28} = +18{,}500$$

$$ab \,\&\, bc = 15{,}000 \times \frac{20}{28} = +10{,}700$$

$$BC \,\&\, CD = 15{,}000 \times \frac{40}{28} = -21{,}400$$

$$cd \,\&\, de = 15{,}000 \times \frac{60}{28} = +32{,}100$$

$$DE \,\&\, EF = 15{,}000 \times \frac{80}{28} = -42{,}800$$

$$ef = 15{,}000 \times \frac{80}{28} = +42{,}800$$

TABLE FOR DEFLECTION AT a, DUE TO LOAD OF 15,000 LBS. AT THIS POINT.

Member	Stress	Area	p	u	l	pul
aB	− 18,500	17.64	−1,050	− 1.230	413	+ 535,000
Bc	+ 18,500	14.70	+ 1,260	+ 1.230	413	+ 640,000
cD	− 18,500	15.88	− 1,160	− 1.230	413	+ 589,000
De	+ 18,500	26.48	+ 700	+ 1.230	413	+ 356,000
ab	+ 10,700	14.70	+ 730	+ 9.714	240	+ 171,000
bc	+ 10,700	14.70	+ 0730	+ 0.714	240	+ 171,000
cd	+ 32,100	14.70	+ 2,190	+ 2.142	240	+ 1,126,000
de	+ 32,100	14.70	+ 2,190	+ 2.142	240	+ 1,126,000
ef	+ 42,800	14.70	+ 2,920	+ 2.856	$\frac{120}{2}$	+ 499,000
BC	− 21,400	14.70	− 1,460	− 1.428	240	+ 355,000
CD	− 21,400	14.70	− 1,460	− 1.428	240	+ 355,000
DE	− 42,800	17.64	− 2,430	− 2.856	240	+ 1,665,000
EF	− 42,800	17.64	− 2,430	− 2.856	$\frac{120}{2}$	+ 416,000
						+ 8,004,000

Then, deflection at $a = \Sigma \frac{pul}{E} = \frac{8{,}004{,}000}{29{,}000{,}000} = 0.28$ in.

Thus the ends require to be lifted 0.28 in. by the screws in order to give a reaction of 15,000 lbs. The balance of the end-deflection (which is equal to 1.4 – 0.28 = 1.12 ins.) will be taken up by shortening members $D E$ and $F G$; and the amount that these members must be shortened is equal to the height that the ends are required to be raised, divided by the coefficient u for

$$D E = \frac{1.12}{2.856} = 0.39 \text{ in.} = \tfrac{3}{8}\text{-in.}$$

NOTE:—The dead-load stresses have been computed for two extreme cases: (I), with no reaction at a; (II), with reaction at a sufficient to lift the end the full amount of the deflection, which has been found to be 0.69 in. Although a reaction of only 15,000 lbs. has been provided by the end-screws, which would lift the ends but 0.28 in., this reaction may be increased by the unequal temperature strains to the full amount provided for in Case (II).

ESTIMATED WEIGHT.

TRUSSES.

$a B$	$= 17.64 \times 34.4 \times \frac{10}{3} \times 4$	= 8,100
$B C D$	$= 14.70 \times 40 \times \frac{10}{3} \times 4$	= 7,850
$D E$	$= 17.64 \times 20 \times \frac{10}{3} \times 4$	= 4,700
$E F$	$= 17.64 \times 10 \times \frac{10}{3} \times 2$	= 1,180
$a b c$	$= 14.70 \times 40 \times \frac{10}{3} \times 4$	= 7,850
$c d e$	$= 14.70 \times 40 \times \frac{10}{3} \times 4$	= 7,850
$e f$	$= 14.70 \times 10 \times \frac{10}{3} \times 2$	= 980
End struts	$= 6.84 \times 17.2 \times \frac{10}{3} \times 4$	= 1,570
Verticals	$= 13.68 \times 28 \times \frac{10}{3} \times 16$	= 20,450
	Forward,	60,530

	Brought forward,	60,530
Bc	$= 14.70 \times 34.4 \times \frac{10}{3} \times 4$	= 6,750
cD	$= 15.88 \times 34.4 \times \frac{10}{3} \times 4$	= 7,350
De	$= 26.48 \times 34.4 \times \frac{10}{3} \times 4$	= 12,200
Center diagonals	$= 2.00 \times 17.2 \times \frac{10}{3} \times 8$	= 920
		87,750
	Details, (40%)	= 35,100
		122,850 lbs.

ONE 20-FT. STRINGER.

Web,	1 plate	36x ⅜	-in.@45.90 lbs.	20 ft.	long	= 910
Flanges,	4 angles	6x6 x$\frac{7}{16}$	" @17.20 "	20 "	"	=1,380
At ends,	4 angles	6x6 x ⅝	" @24.20 "	3 "	"	= 290
Fillers,	4 plates	9x$\frac{7}{16}$	" @13.40 "	2 "	"	= 100
Stiffeners,	10 angles	3x3 x$\frac{5}{16}$	" @ 6.10 "	3 "	"	= 180
						2,860
	Rivet-heads, (3%)					= 80
						2,940 lbs.

ONE 10-FT. STRINGER.

Web,	1 plate	36 x ⅜	-in.@45.90 lbs.	10 ft.	long	= 460
Flanges,	4 angles	3½x3½x ⅜	" @ 8.50 "	10 "	"	= 340
At ends,	4 angles	6 x3½x ⅜	" @11.70 "	3 "	"	= 140
Fillers,	4 plates	6 x ⅜	" @ 7.65 "	2 " 5 ins.	"	= 70
Stiffeners,	4 angles	3 x3 x$\frac{5}{16}$	" @ 6.10 "	3 "	"	= 70
						1,080
	Rivet-heads, (3%)					= 30
						1,110 lbs

ONE STRINGER BRACE-FRAME.

Horizontals,	2 angles	3½x3½x$\frac{5}{16}$	-in.@ 7.10 lbs.	8 ft.	long	=110
Diagonals,	2 angles	3 x2½x$\frac{5}{16}$	" @ 5.50 "	4 " 6 ins.	"	=100
Gussets,	4 plates	10x$\frac{5}{16}$	" @10.62 "	1 "	"	= 40
"	2 plates	10x$\frac{5}{16}$	" @10.62 "	1 " 6 "	"	= 30
						280
	Rivet-heads, (2%)					= 10
						290 lbs.

One End Floorbeam.

				Length.		
Web,	1 plate	42x 3/8	-in. @53.55 lbs.	16 ft. 6 ins.	=	890
Flanges,	4 angles	6x6 x1/2	" @19.60 "	16 " 6 "	=	1,290
End connections,	4 angles	6x6 x5/8	" @24.20 "	2 " 6 "	=	240
Fillers,	4 plates	9x 1/2	" @15.30 "	2 " 6 "	=	150
Brackets,	2 plates	36x 3/8	" @45.90 "	1 "	=	90
"	4 angles	3½x3½x3/8	" @ 8.50 "	1 " 2 "	=	40
"	4 angles	3 x3 x3/8	" @ 7.20 "	2 " 6 "	=	70
Stiffeners,	4 angles	4x3½x3/8	" @ 9.10 "	3 " 6 "	=	130
Fillers,	2 plates	8½x 1/2	" @14.40 "	2 " 6 "	=	70
"						
						2,970
Rivet-heads, (3%)					=	90
						3,060 lbs.

One Intermediate Floorbeam.

				Length.		
Web,	1 plate	42x3/8	-in. @53.55 lbs.	16 ft. 6 ins.	=	890
Flanges,	4 angles	6x6x5/8	" @24.20 "	16 " 6 "	=	1,600
End connections,	4 angles	6x6x5/8	" @24.20 "	2 " 6 "	=	240
Fillers,	4 plates	9 x5/8	" @19.13 "	2 " 6 "	=	190
"	4 plates	13x5/8	" @27.62 "	2 " 6 "	=	280
						3,200
Rivet-heads, (3%)					=	100
						3,300 lbs.

Top Laterals.

6 angles	3x3x5/16	-in. @6.10 lbs.	24 ft. long	=	880
2 angles	3x3x5/16	" @6.10 "	18 " "	=	220
					1,100 lbs.

One Portal Strut.

Top strut,	2 angles	6 x3½x 3/8	-in. @11.70 lbs.	19 ft.	long =	450
Diagonals,	4 angles	3½x3½x 3/8	" @ 8.50 "	11 "	" =	370
"	4 angles	3 x3 x5/16	" @ 6.10 "	5 "	" =	120
Horizontal,	2 angles	3 x3 x5/16	" @ 6.10 "	7 "	" =	90
Gussets,	2 plates	18x 3/8	" @22.96 "	1 " 6 ins.	" =	70
"	2 plates	18x 3/8	" @22.96 "	2 "	" =	90
"	2 plates	12x 3/8	" @15.30 "	1 " 6 "	" =	50
"	2 plates	24x 3/8	" @30.60 "	1 "	" =	60
"	2 plates	12x 3/8	" @15.30 "	2 "	" =	60
						1,360
Rivet-heads, (3%)					=	40
						1,400 lbs.

One Intermediate Top Strut.

Top strut,	2 angles	$3\frac{1}{2}$x3 x$\frac{5}{16}$-in.	@ 6.60 lbs.	19 ft.	long	= 250
Diagonals,	4 angles	3 x3 x$\frac{5}{16}$	" @ 6.10 "	8 "	"	= 200
"	8 angles	3 x3 x$\frac{5}{16}$	" @ 6.10 "	3 "	"	= 150
Horizontal,	2 angles	3 x3 x$\frac{5}{16}$	" @ 6.10 "	10 "	"	= 120
Connections,	4 angles	7 x$3\frac{1}{2}$x$\frac{7}{16}$	" @15.00 "	1 " 6 ins.	"	= 90
Gussets,	2 plates	18x $\frac{3}{8}$	" @22.96 "	1 "	"	= 50
"	3 plates	9 x $\frac{3}{8}$	" @11.48 "	1 "	"	= 30
"	2 plates	9 x $\frac{3}{8}$	" @11.48 "	9 ins.	"	= 20
						910
Rivet-heads, (3%)						= 30
						940 lbs.

Bottom Laterals.

1st panel,	4 angles	3 x3 x $\frac{3}{8}$-in.	@ 7.20 lbs.	24 ft.	long	= 690
2d panel,	4 angles	3 x3 x$\frac{5}{16}$	" @ 6.10 "	24 "	"	= 590
3d panel,	4 angles	$3\frac{1}{2}$x3 x $\frac{3}{8}$	" @ 7.80 "	24 "	"	= 750
4th panel,	4 angles	6 x$3\frac{1}{2}$x $\frac{3}{8}$	" @11.70 "	24 "	"	= 1,120
Center panel,	8 angles	3 x3 x $\frac{3}{8}$	" @ 7.20 "	5 "	"	= 290
Lock angles,	24 angles	3 x3 x $\frac{3}{8}$	" @ 7.20 "	$7\frac{1}{2}$ ins.	"	= 110
"	12 angles	$3\frac{1}{2}$x$3\frac{1}{2}$x $\frac{3}{8}$	" @ 8.50 "	$10\frac{1}{2}$ "	"	= 90
Intersections,	8 plates	18x $\frac{3}{8}$	" @22.96 "	2 ft.	"	= 370
Center panel,	6 plates	12x $\frac{3}{8}$	" @15.30 "	$1\frac{1}{2}$ "	"	= 140
						4,150
Rivet-heads, (2%)						= 80
						4,230 lbs.

One Loading Girder *A*.

Web,	1 plate	42x $\frac{5}{8}$ -in.	@89.25 lbs.	19 ft.	long	= 1,700
Flanges,	4 angles	6x6 x$\frac{1}{2}$	" @19.60 "	19 "	"	= 1,490
"	2 plates	13x $\frac{1}{2}$	" @22.10 "	13 "	"	= 570
End stiffeners,	8 angles	5x$3\frac{1}{2}$x$\frac{1}{2}$	" @13.60 "	$3\frac{1}{2}$ "	"	= 380
Fillers,	4 plates	18x $\frac{1}{2}$	" @30.60 "	$2\frac{1}{2}$ "	"	= 310
Stiffeners over girders B,						
	8 angles	6x6 x$\frac{1}{2}$	" @19.60 "	$3\frac{1}{2}$ "	"	= 550
Stiffeners under stringers,						
	8 angles	5x$3\frac{1}{2}$x$\frac{3}{8}$	" @10.40 "	$3\frac{1}{2}$ "	"	= 290
Fillers,	4 plates	28x $\frac{1}{2}$	" @47.60 "	$2\frac{1}{2}$ "	"	= 480
						5,770
Rivet-heads, (4%)						= 230
						6,000 lbs.

One Brace-Frame between Loading Girders *A*.

Horizontals,	4 angles	3x3x⅜-in.	@ 7.20 lbs.	9	ft. long	= 260
Diagonals,	4 angles	3x3x⅜ "	@ 7.20 "	5	" "	= 140
Gussets,	4 plates	12x⅜ "	@15.30 "	1½	" "	= 90
"	2 plates	12x⅜ "	@15.30 "	2	" "	= 60
						550
Rivet-heads, (2%)						= 20
						570 lbs.

One Loading Girder *B*.

				Length.	
Web,	1 plate	27x¾ -in.	@68.85 lbs.	5 ft. 10 ins.	= 400
Flanges,	4 angles	6x6x¾ "	@28.70 "	5 " 10 "	= 670
End connections,	4 angles	8x8x⅝ "	@32.70 "	2 " 2 "	= 280
Fillers,	4 plates	15x¾ "	@33.26 "	1 "	= 150
Stiffeners,	4 angles	6x6x½ "	@19.60 "	2 " 2 "	= 170
Fillers,	4 plates	15x¾ "	@38.26 "	1 " 6 "	= 230
Bearing for girder A,	1 plate	13x¾ "	@33.15 "	1 " 6 "	= 50
					1,950
Rivet-heads, (3%)					= 50
					2,000 lbs.

One Loading Girder *C*.

				Length.	
Web,	1 plate	27x ⅝ -in.	@57.38 lbs.	8 ft. 6 ins.	= 490
Flanges,	4 angles	6x3½x⅜ "	@11.70 "	8 " 6 "	= 400
End connections,	2 angles	6x6 x⅝ "	@24.20 "	2 " 2 "	= 100
Fillers,	2 plates	15x ⅜ "	@19.13 "	2 " 3 "	= 90
End stiffeners,	2 angles	3x3 x⅜ "	@ 7.20 "	2 " 2 "	= 30
Fillers,	2 plates	6x ⅜ "	@ 7.65 "	1 " 3 "	= 20
					1,130
Rivet-heads, (3%)					= 30
					1,160 lbs.

Circular Girder.

Web,	1 plate	27x ⅜ -in.	@34.42 lbs.	56 ft. 7 ins.	= 1,950
Top flange,	2 angles	6x3½x⅜ "	@11.70 "	56 " 7 "	= 1,320
Bottom flange,	2 angles	6x3½x⅝ "	@18.90 "	56 " 7 "	= 2,140
Web-splice	8 plates	13x ⅜ "	@16.57 "	1 " 3 "	= 160
Flange-splices,	16 plates	5½x ½ "	@ 9.35 "	3 "	= 450
"	4 plates	9x ⅜ "	@11.48 "	3 " 6 "	= 160
Upper tread,	1 plate	6½x1¾ "	@38.68 "	56 " 7 "	= 2,190
					8,370
Rivet-heads, (3%)					= 250
					8,620 lbs.

SUMMARY.

Trusses		= 122,850	122,850	
16 stringers	@ 2,940 =	47,040		
2 stringers	@ 1,110 =	2,220		
10 stringer brace-frames	@ 290 =	2,900		
6 intermediate floorbeams	@ 3,300 =	19,800		
2 end floorbeams	@ 3,060 =	6,120	78,080	
2 portal struts	@ 1,400 =	2,800		
6 intermediate top struts	@ 940 =	5,640		
Top laterals	=	1,100		
Bottom laterals	=	4,230	13,770	
End-lifts	=	6,750		
Latches	=	800	7,550	
				222,250
2 loading girders *A*	@6,000 =	12,000		
2 brace-frames	@ 570 =	1,140		
4 loading girders *B*	@2,000 =	8,000		
8 loading girders *C*	@1,160 =	9,280		
Circular girder	=	8,620	39,040	
Center castings and rollers	=	12,200		
Spacing-rods, etc., for rollers	=	1,700		
Turning-machinery	=	3,260	17,160	
				56,200
Total,				278,450 lbs.

The total weight of steel structure supported by the loading girders = 222,250 lbs.; and the weight per lin. ft. = 222,250 ÷ 170 = 1,300 lbs., which is just equal to the weight assumed for calculating the stresses. The weights of castings, machinery etc., have been taken from similar structures. In order to obtain closer estimates of these items, it would be necessary to make more detailed designs.

CHAPTER VII.

THE DESIGN OF A RAILWAY VIADUCT.

Length of tower-spans, 40 ft.
Length of spans between towers, 60 ft.
Maximum height from top of pedestals to base of rail, 100 ft.
Width at top, 8 ft. c. to c.
Batter of posts transversely, 1 in 6.
Depth of girders, 6 ft.
Ties, 8 x 12 ins. spaced 12 ins. c. to c.
Dead load for 60-ft. spans:

Steel	700
Floor	600
Total	1,300 lbs. per lin. ft.

Dead-load for 40-ft spans:

Steel	600
Floor	600
Total	1,200 lbs. per lin. ft.

Live-load as per specification.

In structures of this class the girders of the long and short spans are usually made the same depth, which arrangement simplifies the details and insures a more pleasing outline than would otherwise be obtained.

For the 60-ft. span, the plate-girder of Chapter II will be used; but the end details will be altered, as shown in Fig. 21.

40-Ft. Span.

END SHEAR:

Dead-load $= 600 \times 20 \quad = 12{,}000$

Live-load, from diagram $= 85{,}000$ (wheel 2 at support)

$$\text{Impact} = \frac{85{,}000^2}{85{,}000 + 12{,}000} = 74{,}500$$

171,500 lbs.

Area required in web-plate $= 171{,}500 \div 10{,}000 = 17.15$ sq. ins.

A 72 x $\frac{3}{8}$-in. plate, = 27 sq. ins., will be used.

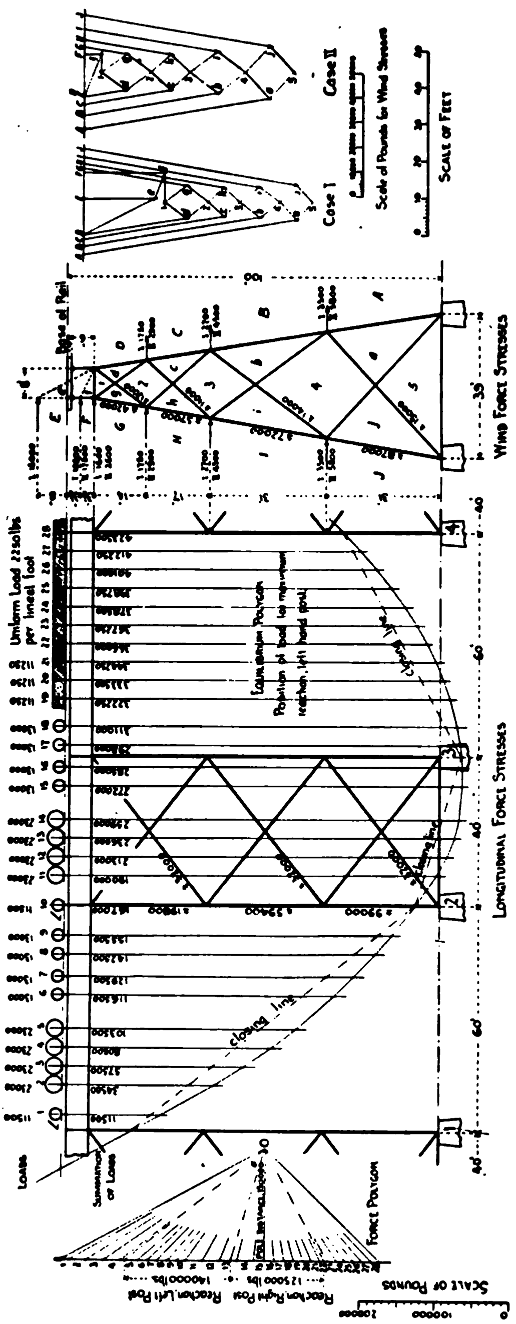


Fig. 20.—Railway Viaduct.

CENTER MOMENT:

$$\text{Dead-load} = \frac{600 \times 40^2}{8} = 120{,}000$$

Live-load, from diagram = 750,000 (wheel 4 at center)

$$\text{Impact} = \frac{750{,}000^2}{750{,}000 + 120{,}000} = 646{,}500$$

1,516,500 ft.-lbs.

Effective depth = 5.75 ft. Flange-stress = 1,516,500 ÷ 5.75 = 263,700 lbs.

Flange-area required = 263,700 ÷ 16,000 = 16.48 sq. ins.

The following section will be used:

$\frac{1}{8}$ of 72 x $\frac{3}{8}$-in. web-plate	= 3.37
2 angles 6 x 6 x $\frac{11}{16}$-in.	= 14.18 (one hole, 1 in. in diam., in each angle)
	17.55 sq ins., net.

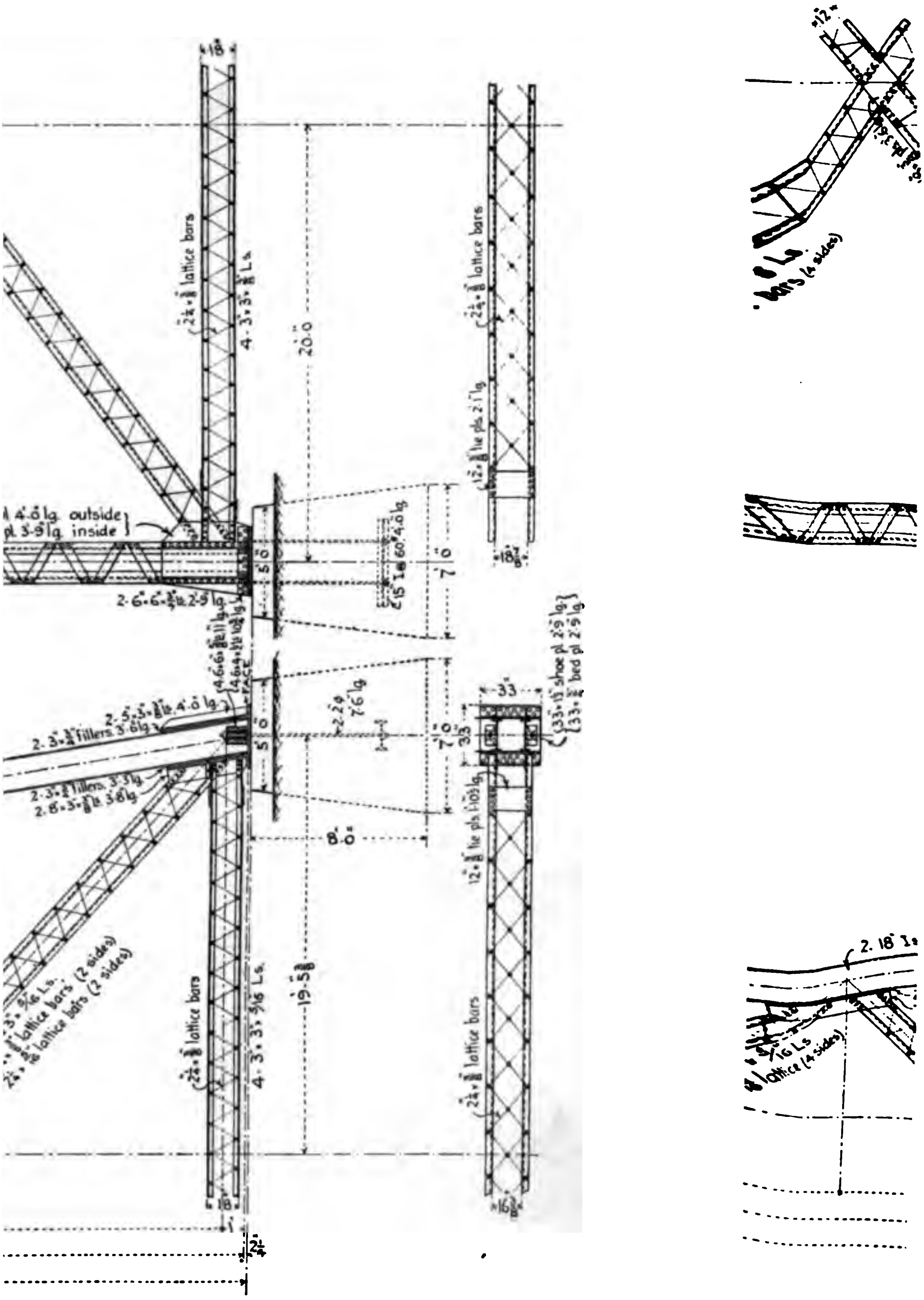
Since there are no flange-plates on the girder, there will be no holes in the horizontal legs of the bottom angles near center of span, and therefore it is only necessary to allow for one rivet-hole in each angle. The web-plates will be spliced at the center, the same as those of the 60-ft. span. For the intermediate stiffeners, two 4 x $3\frac{1}{2}$ x $\frac{3}{8}$-in. angles will be used; and for the top laterals, 3 x 3 x $\frac{5}{16}$-in. angles will be used throughout. There will be three intermediate and two end brace-frames, the same as shown in Fig. 4.

TOWERS.

The towers are composed of four posts, braced transversely and longitudinally. The bracing will all be capable of resisting either tension or compression.

The forces to be considered are as follows: (*a*), dead-load, live-load and impact; (*b*), longitudinal force due to traction of engine, or the sudden application of brakes to a moving train; (*c*), wind force on side of train, girders, and tower.

Dead-Load Stresses.—The dead-load applied at top of post is equal to the reactions of the 60-ft. and 40-ft. spans = (650 × 30) + (600 × 20) = 31,500 lbs.; and the dead-load of one 31-ft. section of post (including longitudinal and transverse bracing) will be assumed at 11,000 lbs. The vertical height of one section



lattice bars
20'-0"
4'-0 lg. outside
pl 3-9 lg. inside
7'-0
5'-0
FACE
8'-0"
19'-5⅝
lattice bars (2 sides)
lattice bars (4 sides)
lattice bars

ıt center)

,500 ÷ 5.75

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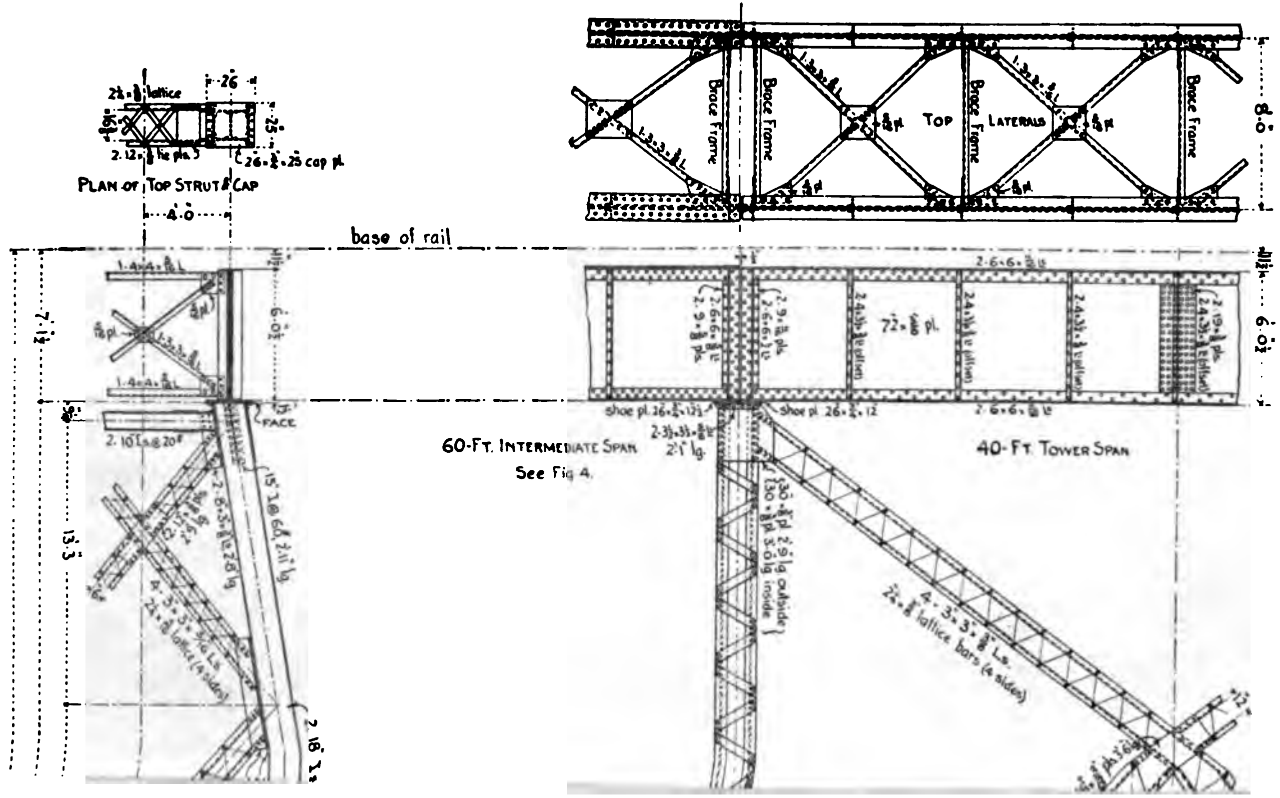

Plan of Top Strut & Cap
base of rail
Brace Frame
Top Laterals
72 × 5/16 pl.
Face
60-Ft. Intermediate Span
See Fig 4.
40-Ft. Tower Span
4 - 3 × 3 × 3/8 Ls.
2 1/2 × 3/8 lattice bars (4 sides)
8·0
6·0 1/2
4·0
7·1 1/2
13·3

of post = 31 ft., and its actual length = 31.33 ft. Then the dead-load stress in a section will be equal to the applied load multiplied by $\frac{31.33}{31}$. Since the batter of posts is 1 in 6, the stress in top transverse strut will be equal to one-sixth of the load on top of post. If it were not for the friction on pedestals, there would be a corresponding tension in the bottom strut. The stresses in this member are indeterminate: it is required principally to equalize the horizontal forces between the two pedestals.

Dead-load stress in top section of post

$$= (31{,}500 + 11{,}000) \times \frac{31.33}{31} = +\ 43{,}000$$

Dead-load stress in middle section of post

$$= (31{,}500 + 22{,}000) \times \frac{31.33}{31} = +\ 54{,}000$$

Dead-load stress in bottom section of post

$$= (31{,}500 + 33{,}000) \times \frac{31.33}{31} = +65{,}000$$

Dead-load stress in top transverse strut $= 31{,}500 \div 6 = +\ 5{,}300$

Live-Load Stresses.—The maximum load on bent No. 2, as well as the maximum load on the whole tower, will be found with the wheel-concentrations placed as shown in Fig. 20, in which the 40-ft. tower-span and the two adjacent 60-ft. spans are placed on the moment-diagram with wheel 10 at the left-hand end of the former. Verticals are dropped from the points of support of the three spans intersecting the equilibrium-polygon, and closing lines are drawn through these intersections. Then, lines drawn through the pole *O* in force-polygon, parallel with these closing lines and intersecting the load-line, will determine the reactions at the points of support. The distance between the intersections of the line parallel with closing line for the left-hand 60-ft. span and the line parallel with closing line for the 40-ft. span, represents the load on post No. 2, which is 140,000 lbs.; and the distance between the intersections of the line parallel with the closing line for the 40-ft. span and the line parallel with the closing line for the right-hand 60-ft. span, represents the load on post No. 3, which is 125,000 lbs.

Live-load stress in all sections of post

$$= 140{,}000 \times \frac{31.33}{31} = +\ 141{,}000$$

Live-load stress in top transverse strut $= 140{,}000 \div 6 = +23{,}300$

Longitudinal-Force Stresses.—The sudden application of brakes to a moving train imparts a longitudinal force, which is equal to the load on tower multiplied by the coefficient of friction between the rails and wheels. This coefficient is assumed to be 2/10. The maximum live-load on the two-posts, as found above, $= 140{,}000 + 125{,}000 = 265{,}000$ lbs.; but from this may be deducted the reactions of the pilot-wheels (1 and 10) because they will have no brakes. These reactions $= 11{,}500\ (1 + 4/60) = 12{,}000$ lbs. Then the longitudinal force on one side of tower $= (265{,}000 - 12{,}000) \times 2/10 = 50{,}600$ lbs., which is assumed to act at top of posts. Since there are two systems of bracing, capable of resisting either tension or compression, the horizontal component of the stress in each will be equal to one-half of this applied force, and the component in line of posts will be equal to the horizontal component multiplied by the length of one section of post and divided by the longitudinal distance between posts. The stress in upper section of posts will be equal to one of these latter components; the stress in the middle section will be equal to three; the stress in the lower section will be equal to five; and the reaction at base will be equal to six. The stress in diagonals will be equal to the horizontal component multiplied by the length of the diagonal and divided by the distance between posts.

The length of diagonals $= \sqrt{31.33^2 + 40^2} = 50.8$ ft.

$$\text{Stress in top section of posts} = 50{,}600 \times \tfrac{1}{2} \times \frac{31.33}{40} = \pm\ 19{,}800$$

Stress in middle section of posts $= 19{,}800 \times 3 = \pm\ 59{,}400$

Stress in lower section of posts $= 19{,}800 \times 5 = \pm\ 99{,}000$

Reaction at base of posts $= 19{,}800 \times 6 = \pm 118{,}800$

$$\text{Stress in diagonals} = 50{,}600 \times \tfrac{1}{2} \times \frac{50.8}{40} = \pm\ 32{,}000$$

Wind-Force Stresses.—There are two cases to be considered according to specification, viz:

Case I, *Structure Loaded.*—A horizontal force of 300 lbs. per lin. ft., applied 8 ft. above base of rail; a horizontal force of 30

lbs. per sq. ft. on the girders and floor; and a horizontal force of 225 lbs. per ft. of height on posts.

Case II, *Structure Unloaded.*—A horizontal force of 50 lbs. per sq. ft. on the girders and floor; and a horizontal force of 375 lbs. per ft. of height on posts.

The concentrations are as follows:

CASE I.

		LBS.
8 ft. above base of rail,	$300 \text{ lbs.} \times \frac{60 \text{ ft.} + 40 \text{ ft.}}{2}$ =	15,000
Center of girders and floor,	$30 \text{ lbs.} \times 7 \text{ ft.} \times \frac{60 \text{ ft.} + 40 \text{ ft.}}{2}$ =	10,500
Top of posts,	$225 \text{ lbs.} \times \frac{14 \text{ ft.}}{2}$ =	1,600
1st panel-point from top of posts,	$225 \text{ lbs.} \times \frac{14 \text{ ft.} + 17 \text{ ft.}}{2}$ =	3,500
2d panel-point from top of posts,	$225 \text{ lbs.} \times \frac{17 \text{ ft.} + 31 \text{ ft.}}{2}$ =	5,400
3d panel-point from top of posts,	$225 \text{ lbs.} \times 31 \text{ ft.}$ =	7,000

CASE II.

Center of girders and floor,	$50 \text{ lbs.} \times 7 \text{ ft.} \times \frac{60 \text{ ft.} + 40 \text{ ft.}}{2}$ =	17,500
Top of posts,	$375 \text{ lbs.} \times \frac{14 \text{ ft.}}{2}$ =	2,600
1st panel-point from top of posts,	$375 \text{ lbs.} \times \frac{14 \text{ ft.} + 17 \text{ ft.}}{2}$ =	5,800
2d panel-point from top of posts,	$375 \text{ lbs.} \times \frac{17 \text{ ft.} + 31 \text{ ft.}}{2}$ =	9,000
3d panel-point from top of posts,	$375 \text{ lbs.} \times 31 \text{ ft.}$ =	11,600

These concentrations are shown in Fig. 20. At the 1st, 2d, and 3d panel points from top of posts, one-half of the concentration is assumed to be applied to the windward post and one-half to the leeward post. In order to facilitate the graphical

computation of the wind-force stresses, imaginary members are supplied to transmit the force applied 8 ft. above base of rail and the force at center of girders and floor to the top of leeward post, as shown in dotted lines.

In the stress-diagram, Case I, the forces *A, B, C, D, E, F, G, H, I, J* are laid off on a horizontal line from left to right. Then, from the points *D* and *E* on load-line, lines are drawn parallel with the dotted members *D e* and *E e*, intersecting in the point *e*. Next, from the point *F* on load-line, and from the point *e* in stress-diagram, lines are drawn parallel with *F f* and *e f*, intersecting in the point *f*. There are now three unknown forces at the top of both windward and leeward posts; but, assuming both systems of bracing to be equally stressed, the difficulty is readily overcome, thus: From the points *D* and *G* on load-line, lines are drawn parallel with members *D d* and *G g*; and, from the point *f* in stress-diagram, a line is drawn parallel with member *f* 1. Now, the point 1 in stress-diagram must be located at such a distance horizontally from *f* that *d* 1 and *g* 1 will be of equal lengths. There will be no further trouble in finishing the diagram. The stresses in both posts of bent, as well as in both systems of bracing, are equal but of opposite character. This diagram gives the maximum wind stresses in posts, and the maximum reaction at base, which latter is equal to the vertical height from point 5 to the load-line = ±92,000 lbs.

Stress-diagram, Case II, which gives the maximum stresses in diagonals, is similarly constructed, except that force *D E* is omitted.

Posts.—The stresses in the top, middle, and bottom sections of posts will now be tabulated.

TOP SECTION:

Dead-load	+ 43,000	
Live-load	+ 141,000	
Impact $= \dfrac{141{,}000^2}{141{,}000+43{,}000} =$	+ 108,200	+ 292,200
Longitudinal	+ 19,800	
Wind	+ 57,000	+ 76,800
		+ 369,000

MIDDLE SECTION:

Dead-load	+ 54,000	
Live-load	+ 141,000	
Impact $= \dfrac{141{,}000^2}{141{,}000+54{,}000} =$	+ 102,000	+ 297,000
Longitudinal	+ 59,400	
Wind	+ 72,000	+ 131,400
		+ 428,400

BOTTOM SECTION:

Dead-load	+ 65,000	
Live-load	+ 141,000	
Impact $= \dfrac{141{,}000^2}{141{,}000+65{,}000} =$	+ 96,500	+ 302,500
Longitudinal	+ 99,000	
Wind	+ 87,000	+ 186,000
		+ 488,500

For the dead-load, live-load, and impact stresses the ordinary unit stress of 16,000 lbs. per sq. in. (reduced by formula for pin-ends) will be used; but, when these are combined with the longitudinal and wind stresses, the unit stress will be increased 25%. Assuming two 18-in. I-beams @ 55 lbs., spaced 15½ ins. c. to c., the least radius of gyration = 7.07 ins. Then $\dfrac{l}{r} = \dfrac{31.33}{7.07} = 4.4$, which corresponds with a permissible unit stress of 12,220 lbs. per sq. in. In the bottom section the dead-load, live-load, and impact stresses = 302,500 lbs., which, at 12,220 lbs = 24.75 sq. ins. required. The total stress in this section = 488,500 lbs., and the permissible unit stress for this case = 12,220 + 25% = 15,270 lbs. Then, 488,500 ÷ 15,270 = 31.99 sq. ins. required. The assumed section of two 18-in. I-beams @ 55 lbs. = 31.86 sq. ins.; and since this is the minimum weight of this size of beam, it will be used throughout.

Longitudinal Bracing.—In order to insure a large degree of rigidity in the structure, the longitudinal diagonals will be

constructed of four 3 x 3 x $\frac{3}{8}$-in. angles, placed back to back but separated 12 ins. in side elevation; and 18 ins. apart in the other direction, as shown in Fig. 21. They will be latticed on four sides. The stress in these members = ±32,000 lbs., their unsupported length. = 25.4 ft., and their least radius of gyration = 6.95 ins. Then $\frac{l}{r} = \frac{25.4}{6.95} = 3.6$, which corresponds with a permissible unit stress of 14,500 lbs. per sq. in. for *square* ends; and the required area = 32,000 ÷ 14,500 = 2.2 sq. ins. The area of four 3 x 3 x $\frac{3}{8}$-in. angles = 8.44 sq. ins., which is much greater than that required for the stress; but it is undesirable to use smaller angles on account of the riveting, and the specification only permits $\frac{5}{16}$-in. metal in wind bracing, lattice-bars, and stiffeners.

The bottom longitudinal strut will be of the same material as the diagonals; but, on account of its greater unsupported length, the angles will be 18 ins. back to back, vertically, as shown.

Transverse Bracing.—In the top strut the stresses are:

Dead-load	+ 5,300
Live-load	+23,300
Impact $= \frac{23,300^2}{23,300+5,300} =$	+18,900
	+ 47,500 lbs.

The stress due to wind force is not included in the above, because it is tension and only tends to diminish the compression due to dead- and live-loads. For this member two 10-in. channels @ 20 lbs. will be used, with the webs placed vertically.

The diagonals and bottom strut will be similar to the longitudinal bracing, except that $\frac{5}{16}$-in. metal will be used, and the angles of the shorter members near the top of bent will be closer together, as shown in Fig. 21. If these short members were as wide as the longer ones, they would look out of proportion.

Bearing Plates.—In figuring the bearing area required, the unit pressure on concrete for dead-load, live-load, and impact will be taken at 400 lbs. per sq. in.; but, when these are com-

bined with longitudinal and wind loads, the unit pressure will be increased 25%. The total reaction is as follows:

Dead-load	$= 31{,}500 + 33{,}000$	$= 64{,}500$	
Live-load		$= 140{,}000$	
Impact	$= \dfrac{140{,}000^2}{140{,}000 + 64{,}500}$	$= 95{,}800$	300,300
Longitudinal		$= 118{,}800$	
Wind		$= 92{,}000$	210,800
			lbs. 511,100

Then, $300{,}300 \div 400 = 751$ sq. ins.; and $511{,}100 \div 500 = 1{,}022$ sq. ins., which latter is the area required in bed-plates. In Fig. 21 the plates are 33 x 33 ins. = 1,089 sq. ins. The upper, or shoe-plate, which is riveted to the post, is $1\frac{1}{2}$ ins. thick; and the lower, or bed-plate, $\frac{3}{4}$-in. thick.

Anchors.—The greatest tension on the anchor-bolts of the windward post of bent No. 3 will occur when the uniform live-load of 2,250 lbs. per lin. ft. (on each rail) covers span 1–2, and extends about three-fourths of the distance from bent No. 2 to bent No. 3. Assuming this condition of loading,

$$\text{Live load on post No. 2} = (2{,}250 \times 30) + \left(2{,}250 \times 30 \times \frac{25}{40}\right) = 109{,}700$$

$$\text{Live-load on post No. 3} = \left(2{,}250 \times 30 \times \frac{15}{40}\right) = 25{,}300$$

Total live-load on one side of tower 2–3 = 135,000 lbs.

Longitudinal force at top of posts = $135{,}000 \times 2/10 = 27{,}000$ lbs.

Vertical reaction at the foot of post No. 3 due to this force =

$$27{,}000 \times \frac{94}{40} = -63{,}400 \text{ lbs.}$$

The wind force, 8 ft. above base of rail, for bent No. 3 will be $300 \text{ lbs.} \times 30 \text{ (ft.)} \times \frac{15}{40} = 3{,}400$ lbs. Now the difference between this force and the corresponding force of Case I, as shown in Fig. 20, = 15,000–3,400 = 11,600 lbs.; and the vertical reaction

at the foot of windward post will be equal to that obtained from stress diagram, Case I, less that due to this difference = 92,000 − $\left(11{,}600 \times \frac{108}{39}\right)$ = −59,900 lbs. The reactions will now be summarized.

Longitudinal force	= − 63,400	
Wind force	= − 59,900	− 123,300
Dead-load	= + 64,500	
Live-load	= + 25,300	+ 89,800
		− 33,500 lbs.

Thus the maximum tension on the anchor-bolts = 33,500 lbs., which, @ 10,000 lbs. per sq. in. = 3.35 sq. ins. required. The above low unit stress is employed on account of the probable deterioration from rust. Two bolts, 2 ins. in diameter, will be used, the net area of which (allowing for threads) = 4 sq. ins. (about). The anchors should have attached to them, at their lower extremities, a heavy piece of beam or channel; and the whole should be built into the concrete. It is not advisable to depend on the adhesion of concrete to the rods in structures of this kind, as it may be destroyed by vibration.

Assuming concrete to weigh at least 150 lbs. per cu. ft., the volume required = 33,500 ÷ 150 = 223 cu. ft. A pedestal 5 ft. square at the top, 7 ft. square at the bottom, and 8 ft. deep would contain 290 cu. ft.

Details.

Fig. 21 is a detail drawing showing one-half of a bent, and one-half of the longitudinal elevation of tower, also one end of a 60-ft. span.

The 40-ft. span will be riveted at both ends to the tops of posts, but the 60-ft. span will be riveted to the posts at one end only. At the opposite end, there will be slotted holes in the shoe-plates to provide for expansion, and bolts with nuts and washers will be used to connect the shoes with the caps of posts. The 60-ft. span (excepting the altered end details) and the intermediate brace-frames of the 40-ft. span will be the same as shown in Fig. 4.

In the tower the vertical spacing of the transverse and longitudinal bracing is varied slightly from that shown in Fig. 20,

in order to arrange efficient connections for the top and bottom struts. The various thicknesses of lattice-bars are determined by the rule that their unsupported length shall not exceed fifty times their thickness. At the foot of posts, substantial brackets are provided to take the pull of the anchor-bolts. The field-splices in posts are so arranged that the upper sections can be dropped into place over the lower sections, which is an important consideration from the erecter's point of view. The 8 x 3 x $\frac{3}{8}$-in. angles used in this design are rolled in Scotland. If they could not be obtained, it would be necessary to use small angles and gusset plates to connect the transverse bracing to posts.

In order to facilitate the shopwork by having as many parts alike as possible, the upper sections of the shorter towers should be made like those of the highest tower, and the difference in height allowed for in the bottom section. This rule does not apply to towers near the ends of the viaduct, which may be shorter than a single section of the highest. In these short towers it may be advisable to decrease the size of posts.

Estimated Weight.

Referring to Fig. 4, and taking into account the altered detail at ends, as shown in Fig. 21, the weight of the 60-ft. girders is estimated as follows:

Two 60-ft. Girders.

Flanges.					
8 angles,	6x6 x$\frac{5}{8}$-in.	@24.20 lbs.	60	ft. long	= 11,620
2 plates,	15x $\frac{5}{8}$	" @31.88 "	60	" "	= 3,830
2 plates,	15x $\frac{5}{8}$	" @31.88 "	45.75	" "	= 2,920
4 plates,	15x $\frac{5}{8}$	" @31.88 "	31.75	" "	= 4,050
Webs.					
2 plates,	72x $\frac{3}{8}$	" @91.80 "	60	" "	= 11,020
Web-splices.					
4 plates,	19x $\frac{3}{8}$	" @24.23 "	5	" "	= 480
Stiffeners.					
8 angles,	6x6 x$\frac{5}{8}$	" @24.20 "	6	" "	= 1,160
36 angles,	4x3$\frac{1}{2}$x$\frac{3}{8}$	" @ 9.10 "	6	" "	= 1,970
Fillers,					
8 plates,	9x $\frac{5}{8}$	" @19.13 "	5	" "	= 760
Shoes.					
4 plates,	26x $\frac{3}{4}$	" @66.30 "	1	ft. "	= 270
					38,080
Rivet-heads, (4%)					= 1,520
					39,600 lbs.

SUMMARY FOR 60-FT. SPAN.

2 Girders		=	39,600
Top Laterals,		=	1,410
2 End Brace-Frames	@290	=	580
4 Intermediate Brace-Frames	@270	=	1,080
			42,670 lbs.

TWO 40-FT. GIRDERS.

Flanges.						
8 angles,	6x6 x$\frac{11}{16}$-in.	@26.50 lbs.	39 ft. 11 ins.	long	=	8,470
Webs.						
2 plates,	72x $\frac{3}{8}$	" @91.80 "	39 " 11 "	"	=	7,330
Web-splices.						
4 plates,	19x $\frac{3}{8}$	" @24.23 "	5 "	"	=	480
Stiffeners.						
8 angles,	6x6 x $\frac{1}{2}$	" @19.60 "	6 "	"	=	940
28 angles,	4x3$\frac{1}{2}$x $\frac{3}{8}$	" @ 9.10 "	6 "	"	=	1,530
Fillers.						
8 plates,	9x$\frac{11}{16}$	" @21.04 "	5 "	"	=	840
Shoes.						
4 plates,	26x $\frac{3}{4}$	" @66.30 "	1 ft.	"	=	270
						19,860
Rivet-heads, (4%)					=	790
						20,650 lbs.

TOP LATERALS, 40-FT. SPAN.

Main Material.						
8 angles,	3x3 x$\frac{5}{16}$-in.	@ 6.10 lbs.	11 ft.	long	=	540
Connections.						
16 plates,	12x$\frac{5}{16}$	" @ 12.75 "	1 " 6 ins.	"	=	310
Splices						
4 plates,	18x$\frac{5}{16}$	" @ 19.13 "	1 " 6 "	"	=	110
						960
Rivet-heads, (2%)					=	20
						980 lbs.

SUMMARY FOR 40-FT. SPAN.

2 Girders,		=	20,650
Top Laterals,		=	980
2 End Brace-Frames	@290	=	580
3 Intermediate Brace-Frames	@270	=	810
			23,020 lbs.

The weight per lin. ft. of the 40-ft. span = 23,020 ÷ 40 = 575 lbs., which is 25 lbs. less than that assumed for the calculations.

ONE POST.

			LENGTH.				
Main Material.							
2 18-in. I-beams		@55.00 lbs.	94 ft.		=	10,340	10,340
At cap							
1 plate,	26x $\frac{3}{4}$ -in.	@66.30 "	2 "	1 ins.	=	140	
1 plate,	30x $\frac{3}{8}$ "	@38.25 "	2 "	9 "	=	100	
1 plate,	30x $\frac{3}{8}$ "	@38.25 "	3 "		=	110	
2 angles,	$3\frac{1}{2}$x$3\frac{1}{2}$x $\frac{3}{8}$ "	@13.60 "	2 "	1 "	=	60	
1 15-in. I-beam		@60.00 "	2 "	11 "	=	180	
2 angles,	8x3 x $\frac{3}{8}$ "	@13.60 "	2 "	8 "	=	70	660
At base.							
1 plate,	33x$1\frac{1}{2}$ "	@168.32 "	2 "	9 "	=	460	
1 plate,	33x $\frac{3}{4}$ "	@ 84.16 "	2 "	9 "	=	230	
1 plate,	37x $\frac{3}{8}$ "	@ 47.18 "	4 "		=	190	
1 plate,	37x $\frac{3}{8}$ "	@ 47.18 "	3 "	9 "	=	180	
2 angles,	6x6 x $\frac{3}{4}$ "	@ 28.70 "	2 "	9 "	=	160	
2 angles,	5x3 x $\frac{3}{8}$ "	@ 9.80 "	4 "		=	80	
2 angles,	8x3 x $\frac{3}{8}$ "	@ 13.60 "	3 "	8 "	=	100	
2 flats	3x $\frac{3}{4}$ "	@ 7.65 "	3 "	6 "	=	50	
2 flats,	3x $\frac{3}{4}$ "	@ 7.65 "	3 "	3 "	=	50	
4 angles,	6x6 x $\frac{5}{8}$ "	@ 24.20 "		11 "	=	90	
4 angles,	6x4 x $\frac{1}{2}$ "	@ 16.20 "		$10\frac{3}{4}$ "	=	60	
1 15-in. I-beam		@ 60.00 "	4 "		=	240	
2 rounds, 2 ins. diam.,		@ 10.68 "	7 "	6 "	=	160	
4 nuts,		@ 4.50 "			=	20	2,070
Splice-plates.							
8 plates	32x $\frac{3}{8}$ "	@40.80 "	4 "	3 "	=	1,390	
8 plates,	7x $\frac{3}{8}$ "	@ 8.93 "	1 "	4 "	=	90	
Tie-plates.							
1 plate,	22x $\frac{3}{8}$ "	@28.05 "	5 "	3 "	=	150	
1 plate,	22x $\frac{3}{8}$ "	@28.05 "	1 "	3 "	=	40	
1 plate,	22x $\frac{3}{8}$ "	@28.05 "	1 "	$10\frac{1}{2}$ "	=	50	
Connections.							
2 angles,	8x3 x $\frac{3}{8}$ "	@13.60 "	5 "	3 "	=	140	
2 angles,	8x3 x $\frac{3}{8}$ "	@13.60 "	5 "	6 "	=	150	
2 angles,	8x3 x $\frac{3}{8}$ "	@13.60 "	5 "	9 "	=	160	
Latticing.							
94 flats,	5x$\frac{7}{16}$ "	@ 7.44 "	2 "	1 "	=	1,560	3,730
							16,800
Rivet-heads, (3%)							500
							17,300 lbs.

One Set Transverse Bracing.

			Length.			
Top strut.						
2 10-in. channels		@20.00 lbs.	6 ft. 6 ins.	=	260	
4 plates,	12x $\frac{3}{8}$	-in. @15.30 "	1 " $9\frac{1}{2}$ "	=	110	
8 flats,	$2\frac{1}{4}$x $\frac{3}{8}$	" @ 2.87 "	2 " 6 "	=	60	430
Top panel.						
4 angles,	3x3 x$\frac{5}{16}$	" @ 6.10 "	14 "	=	340	
4 angles,	3x3 x$\frac{5}{16}$	" @ 6.10 "	5 "	=	120	
4 angles,	3x3 x$\frac{5}{16}$	" @ 6.10 "	8 "	=	200	
2 plates,	12x $\frac{3}{8}$	" @15.30 "	2 " 9 "	=	80	
12 plates,	11x $\frac{3}{8}$	" @14.03 "	1 " 3 "	=	210	
68 flats,	$2\frac{1}{4}$x$\frac{5}{16}$	" @ 2.39 "	1 " 2 "	=	190	
68 flats,	$2\frac{1}{4}$x$\frac{5}{16}$	" @ 2.39 "	1 " 5 "	=	230	1,370
1st panel from top.						
4 angles,	3x3 x$\frac{5}{16}$	" @ 6.10 "	20 "	=	490	
4 angles,	3x3 x$\frac{5}{16}$	" @ 6.10 "	7 " 6 "	=	180	
4 angles,	3x3 x$\frac{5}{16}$	" @ 6.10 "	11 " 6 "	=	280	
2 plates,	15x $\frac{3}{8}$	" @19.12 "	3 "	=	110	
12 plates,	11x $\frac{3}{8}$	" @14.03 "	1 " 3 "	=	210	
104 flats,	$2\frac{1}{4}$x$\frac{5}{16}$	" @ 2.39 "	1 " 5 "	=	350	
104 flats,	$2\frac{1}{4}$x$\frac{5}{16}$	" @ 2.39 "	1 " 5 "	=	350	1,970
2d panel from top.						
4 angles,	3x3 x$\frac{5}{16}$	" @ 6.10 "	35 "	=	850	
4 angles,	3x3 x$\frac{5}{16}$	" @ 6.10 "	13 "	=	320	
4 angles,	3x3 x$\frac{5}{16}$	" @ 6.10 "	20 " 6 "	=	500	
2 plates,	18x $\frac{3}{8}$	" @22.95 "	3 " 6 "	=	160	
12 plates,	11x $\frac{3}{8}$	" @14.03 "	1 " 3 "	=	210	
160 flats,	$2\frac{1}{4}$x $\frac{3}{8}$	" @ 2.87 "	1 " 9 "	=	800	
160 flats,	$2\frac{1}{4}$x$\frac{5}{16}$	" @ 2.39 "	1 " 6 "	=	570	3,410
Bottom panel.						
4 angles,	3x3 x$\frac{5}{16}$	" @ 6.10 "	42 "	=	1,020	
4 angles,	3x3 x$\frac{5}{16}$	" @ 6.10 "	17 " 3 "	=	420	
4 angles,	3x3 x$\frac{5}{16}$	" @ 6.10 "	23 " 3 "	=	570	
2 plates,	18x $\frac{3}{8}$	" @22.95 "	3 " 3 "	=	150	
12 plates,	11x $\frac{3}{8}$	" @14.03 "	1 " 3 "	=	210	
200 flats,	$2\frac{1}{4}$x $\frac{3}{8}$	" @ 2.87 "	1 " 9 "	=	1,000	
200 flats,	$2\frac{1}{4}$x$\frac{5}{16}$	" @ 2.39 "	1 " 6 "	=	720	4,090
Bottom strut.						
4 angles,	3x3 x$\frac{5}{16}$	" @ 6.10 "	37 "	=	960	
4 plates,	12x $\frac{3}{8}$	" @15.30 "	1 " $10\frac{1}{2}$ "	=	120	
100 flats,	$2\frac{1}{4}$x $\frac{3}{8}$	" @ 2.87 "	1 " 8 "	=	480	
60 flats,	$2\frac{1}{4}$x $\frac{3}{8}$	" @ 2.87 "	2 " 10 "	=	490	2,050
						13,320
Rivet-heads, (4%)				=		530
						13,850 lbs.

ONE SET LONGITUDINAL BRACING.

Diagonals.				LENGTH.			
12 angles,	3x3 x$\frac{3}{8}$-in.	@ 7.20 lbs.	48 ft.		=	4,140	
24 angles,	3x3 x$\frac{3}{8}$ "	@ 7.20 "	23 "		=	3,970	
6 plates,	18x $\frac{3}{8}$ "	@22.95 "	3 "	6 ins.	=	480	
672 flats,	2$\frac{1}{4}$x $\frac{3}{8}$ "	@ 2.87 "	1 "	9 "	=	3,370	
672 flats,	2$\frac{1}{4}$x $\frac{3}{8}$ "	@ 2.87 "	1 "	8 "	=	3,210	
36 plates,	11x $\frac{3}{8}$ "	@14.03 "	1 "	5$\frac{1}{2}$ "	=	740	15,910
Bottom strut.							
4 angles,	3x3 x$\frac{3}{8}$ "	@ 7.20 "	38 "	2 "	=	1,100	
4 plates,	12x $\frac{3}{8}$ "	@15.30 "	2 "	1 "	=	130	
104 flats,	2$\frac{1}{4}$x $\frac{3}{8}$ "	@ 2.87 "	1 "	8 "	=	500	
60 flats,	2$\frac{1}{4}$x $\frac{3}{8}$ "	@ 2.87 "	3 "		=	520	2,250
							18,160
Rivet-heads, (4%)					=		720
							18,880 lbs.

SUMMARY FOR ONE TOWER.

4 Posts,	@ 17,300 =	69,200
2 Sets Transverse Bracing,	@ 13,850 =	27,700
2 Sets Longitudinal Bracing,	@ 18,900 =	37,800
		134,700 lbs.

Thus, the weight of one 31-ft. section of post with bracing = $\frac{134,700}{4\times3}$ = 11,200 lbs., which is practically equal to that assumed for dead-load in the calculations.

With the above estimated weights of the 60-ft. and 40-ft. spans and that of the highest tower, it will be a simple matter to obtain the weight of the complete viaduct. Referring to the estimate for post 94 ft. long, it will be found that the details at the top = 660 lbs., the details at the base = 2,070 lbs., and the intermediate details = 3,730 lbs. = 40 lbs. per lin. ft. With these figures, the weight of a post 63 ft. long may be estimated, thus:

ONE POST, 63-FT. LONG.

2 18-in. I-beams @ 55 lbs., 63 ft. long	=	6,930
Details at cap	=	660
Details at base	=	2,070
Intermediate details, 40 lbs. × 63 ft.	=	2,520
		12,180
Rivet-heads, (3%)	=	360
		12,540 lbs.

The transverse bracing for this post would be the same as for the 94-ft. post, except that the bottom panel of diagonals would be omitted, and the bottom strut would be about 27 ft. long instead of 37 ft. Then:

ONE SET TRANSVERSE BRACING FOR 63-FT. POSTS.

Top strut	= 430
Diagonals, top panel	= 1,370
Diagonals, 1st panel from top	= 1,970
Diagonals, 2d panel from top	= 3,410
Bottom strut, $= 2{,}050 \times \frac{27}{37}$	= 1,500
	8,680
Rivet-heads (4%)	= 340
	9,020 lbs.

For the longitudinal bracing the weight of diagonals would be equal to two-thirds of that for the 94-ft. posts, but the bottom strut would be the same. Then:

ONE SET LONGITUDINAL BRACING FOR 63-FT. POSTS.

Diagonals	$= 15{,}910 \times \frac{2}{3}$	= 10,460
Bottom strut		= 2,250
		12,710
Rivet-heads, (4%)		= 510
		13,220 lbs.

In cases where the bottom panel is of different height from that of the highest tower, the lengths of the transverse and longitudinal diagonals may be determined, approximately, by scale from a single line diagram, and their weights obtained by proportion.

CHAPTER VIII.

ADDITIONAL TYPES OF STEEL RAILWAY BRIDGES.

It would be impracticable to treat in detail all of the various types of steel railway bridges; but some of the more important variations from the examples which have been considered in the previous chapters will be dealt with briefly, in order to show how the principles therein set forth and illustrated may be applied to other problems.

THROUGH PLATE-GIRDERS.

Through plate-girders are used in locations where the allowable height from top of masonry to base of rail is not great enough for a deck plate-girder. The width c. to c. of girders is governed by the clearance-diagram of specification. The floor is usually supported by stringers and floorbeams, as in truss bridges; but sometimes the ties rest directly on continuous shelf-angles, riveted to the webs of girders. When the shallowest possible floor is required, owing to limited headroom, the floorbeams are sometimes spaced about 15 ins. c. to c. and support the rails directly, the rails and guard-rails being fastened to a $\frac{3}{8}$-in. plate from six to seven feet wide and of the full length of floor, riveted to the top flange of floorbeams; or a ballast floor may be used constructed of trough-sections, in which case the rails are fastened to ordinary ties in the same manner as on the permanent roadbed. Ballast floors, however, are not confined to locations where the headroom is limited; but they are used on all kinds of railway bridges. Although expensive to construct, the cost of maintaining them is much less than that of ordinary floors; and vibration and noise are greatly reduced, owing to the ballast which forms a cushion between the ties and the bridge-floor. The stresses in closely-spaced floorbeams and in trough-sections are somewhat indeterminate, for they depend on the relative stiffness of the rails and floorbeams. It is usually customary, however, to assume that the maximum wheel-con-

centration is distributed over from three to four feet of the floor longitudinally, and the beams or trough-sections are designed accordingly.

A through plate-girder span in which the ties are supported on continuous shelf-angles riveted to the webs, or one with closely-spaced floorbeams or trough-sections, also riveted to the webs, is designed similarly to a deck plate-girder (Chapter II). The only difference to be noted is that, since there is no load applied to the top or bottom flange-angles, the flange-rivets require to be proportioned for the longitudinal shearing stresses only, and the maximum allowable pitch of rivets would be 6 ins. A through span without floorbeams should have latticed struts about ten feet apart and as deep as the distance from bottom of girders to base of rail will permit. The top flanges of the girders should be braced to these struts by gusset plates. In this form of girder, the laterals are most efficient when connected by gusset plates to the shelf-angles which support the ties. With closely-spaced floorbeams or trough-sections, no struts or laterals are required; but there should be gusset plates about ten feet apart to stiffen the top flanges.

In through plate-girder spans having stringers and floorbeams, the latter should not be more than fifteen feet apart, and they should have gusset plates to stiffen the top flanges, as before mentioned. The floorbeams should not be connected to the girders by the stiffener-angles, as such connections are unsymmetrical and induce serious secondary stresses. The end connections should be made of two heavy angles, similarly to those of the floorbeams shown in Fig. 12. The inside stiffener-angles of the girders at these points should extend from their top flanges to the top of floorbeams; and the gusset plates should be riveted to these stiffener-angles and to short angles riveted to the top of floorbeams at their ends. To determine the lengths of flange-plates for this class of girder, the maximum moments and resulting flange-areas required should be computed at each floorbeam connection. Then, using these areas as ordinates and the panel lengths (or distances between floorbeam connections) as abscissas, the curve of required areas should be constructed, from which the lengths of cover-plates may be scaled, as in Fig. 3. To determine the rivet-spacing in flanges, the maximum shear in each panel should be computed in the same manner as for a

trussed girder. Since the shear from one panel-point to the next is constant, the longitudinal shear at the flanges between these points is also constant, and the rivet-spacing should be uniform. As there are no vertical loads on the flange-rivets, they are required to be proportioned for the longitudinal shear only.

Through Warren Girders.

Through Warren-girder bridges, as usually constructed, are not deep enough to permit of top-lateral bracing. In outline, the trusses are similar to that shown in Fig. 5, except the vertical end-posts and the end sections of top chord are omitted. The loads are applied at the lower panel-points, where the floorbeams are located. The floorbeams should be riveted to the vertical posts, which members are not subject to any direct stresses, but are required to stiffen the top chord vertically as well as horizontally. In order to give increased lateral stability to the top chord, gusset plates should also be provided, connected to the top flange of floorbeams and extending as high up on posts as the clearance-diagram of specification will allow. Sometimes outside braces (or wings) are provided for this purpose, but they are not here recommended, as they are of very little service and by no means ornamental.

Deck Spans with Floorbeams and Stringers.

In outline the trusses for this class of bridge are usually similar to that shown in Fig. 9, except vertical end-posts are provided and the top chord extends the full length of span. Sometimes the floorbeams are supported directly on the top chord; but more frequently they are riveted to the vertical posts directly below the top chord. The top laterals should extend from end to end of the bridge, at which points substantial vertical brace-frames are required to transmit the total wind forces to the abutments. At the intermediate panel-points, light brace-frames should be provided to give additional stiffness. The loads are assumed to be applied at the upper panel-points; but in other respects the design is similar to that of the through span (Chapter IV).

Pin-Connected Spans.

For short spans pin-connected designs have generally gone out of favor on account of their lack of rigidity: but for spans of

over 200 ft. in length they are still considered to be the most suitable form of construction. They weigh less than riveted spans, as there is a considerable saving in the tension members, which are made of eyebars; and the cost of erecting them is less, on account of the smaller number of field-rivets to be driven. The main members of pin-connected spans are designed in the same manner as those of the riveted designs given herewith, except that the pin-end formula should be used in determining the sections of posts. The details of the joints, however, are different. For method of proportioning pins and pin-plates, the reader is referred to the article on pin-connected spans in a previous work of the author's, entitled "Bridge and Structural Design." Stiff lateral and sway-bracing is now used in structures of this kind, similar to that employed in riveted designs.

In addition to the types of bridges already mentioned, there are suspension bridges, which are only used for very long spans; cantilever bridges, which are used either for long spans or in locations where it is difficult to erect falsework; arches, which are most suitable for crossing ravines; and bascule bridges, which are principally used in cities, their main advantage over swing-spans being that they require less room. These types, however, are evidently beyond the scope of this work, as each would require a volume to do it justice.

CHAPTER IX.

THE LATTICING OF COMPRESSION MEMBERS.

The failure of the Quebec Bridge, which occurred on August 29, 1907, has emphasized the necessity of obtaining some logical and simple method of proportioning the lattice-bars of compression members, and has led to much serious consideration and study of the stresses therein. Hitherto there has been but little theoretical knowledge on this important subject, and there are probably few, if any, existing bridges for which the latticing has been proportioned otherwise than by rule-of-thumb. Specifications usually require that the width of lattice-bars shall vary according to the depth of the channels of which the compression member is composed, and that their thickness shall not be less than a certain fraction of their length c. to c. of rivets; that single latticing shall be set at an angle of 60° with the longitudinal axis of column, and double latticing at an angle of 45°. Thus no account is taken of the sectional area of the member, and a light one would be latticed with the same size of bars as a heavier one of the same general dimensions, which does not seem reasonable. Yet these rules, being the result of long experience, are usually found to give results which are not far astray when applied to ordinary cases; but for members of large sectional area in comparison with the general dimensions of their cross-section, the size of latticing obtained thereby may, in some cases, be inadequate; and, for exceptionally large compression members, the designer has been left almost entirely to his own resources, with little to guide his judgment in the matter.

Recently, however, there have been many communications in the technical press relating to this subject, with various suggestions for dealing with the problem. In *Engineering News* of Nov. 7, 1907, Mr. C. T. Morris, Assoc. M. Am. Soc. C. E., Associate Professor of Civil Engineering, Ohio State University, gives a very clear and consistent method; but which requires

to be somewhat modified, as indicated by the author in a letter published in *Engineering News*, Dec. 12, 1907. This method is easily applied; it may be used in connection with any form of column formula; and by it the latticing can be proportioned so as to develop the fiber-stress due to bending assumed in the particular formula adopted. The explanation of this method, in its modified form, is as follows:

Column formulas are based on the assumption that the column bends under load, thus increasing the fiber-stress on the concave side and diminishing it by the same amount on the convex side. Then, in order that the maximum fiber-stress will not exceed that allowed for a short block in compression, the permissible unit-stress to be used in determining the area of cross-section must be less than that allowed for the short block by an amount equal to the additional fiber-stress due to flexure. In other words, the permissible unit-stress at the center of gravity of the section should be such that the maximum fiber-stress on the outer fibers will not exceed that allowed for a short block. In a column composed of two ribs, or channels, connected together by lattice-bars and loaded symmetrically, one-half of the total maximum difference in stress between the two ribs at the middle-point of column must be transmitted from one rib to the other by the lattice-bars; and the amount of stress to be transmitted is equal ta the difference between the average unit-stress at the center of gravity of the section and that at the center of gravity of either rib multiplied by the area of one rib, the unit-stress at the center of gravity of a rib being proportional to the distance of this point from the center of gravity of the whole section. For a pin-end column, which deflects in a simple curve, this stress must be transmitted in one-half the length of the column; but, in a fixed-end column, it must be transmitted in one-quarter of its length, as the stress in the two ribs must be equal at the points of contraflexure, which are at the quarter-points, where the bending moment is zero. Then, for a pin-end column, the increment of stress per lin. ft. of column to be transmitted by the lattice-bars is equal to the total amount of stress to be transmitted divided by one-half the length of column; and, for a square-end column, it is equal to the total stress to be transmitted divided by one-quarter of its length.

The above theory will now be treated mathematically, using the following notation:

l = length of column in feet.
A = area of cross-section.
r = radius of gyration.
x = distance from center of gravity of cross-section to center of gravity of either rib.
n = distance from center of gravity of cross-section to outer fibers.
p = permissible compression for short blocks, = maximum outer fiber stress.
p_1 = permissible average compression per sq. in., by column formula.
d = difference between permissible average compression per sq. in. at center of gravity of section and that at center of gravity of either rib,

$$= (p - p_1)\frac{x}{n}.$$

i = increment of stress per lin. ft. of column to be transmitted by the lattice-bars.
s = stress in each lattice-bar.
y = longitudinal distance in feet covered by one lattice-bar.
α = angle which the lattice-bars make with the longitudinal axis of column.

Then i (for pin-end columns) $= \dfrac{\frac{1}{2}Ad}{\frac{1}{2}l} = \dfrac{A\,d}{l}$ (1)

i (for fixed-end columns) $= \dfrac{\frac{1}{2}Ad}{\frac{1}{4}l} = \dfrac{2\,Ad}{l}$ (2)

s (for single latticing on two sides of column)

$$= \frac{i\,y \sec \alpha}{2} \quad (3)$$

s (for double latticing on two sides of column)

$$= \frac{i\,y \sec \alpha}{4} \quad (4)$$

For determining the stress to be transmitted by the lattice-bars of short compression members (in which the length is less than thirty-six times the radius of gyration) p_1 should be computed in the same manner as for longer columns, notwithstanding the instructions to the contrary in the specification, Chap. I.

Lattice-bars are subject to either tension or compression, and should be designed for the latter condition, using the column formula for fixed ends.

The latticing of some of the compression members in the bridges treated of in the previous chapters will now be considered.

Taking for the first example the top chord member $C\,D$ of the 150-ft. span, dealt with in Chap. IV, which is made up as follows:

1 cover-plate	$21\text{x}\frac{7}{16}$ ins.	=	9.19
2 web-plates	$18\text{x}\ \frac{3}{8}$ "	=	13.50
4 flange-angles	$6\text{x}3\frac{1}{2}\text{x}\frac{7}{16}$ "	=	15.88
			38.57 sq. ins.

the web-plates being 14 ins. out to out, and the 6-in. legs of angles vertical; it will only be necessary to consider the lower part of the section, as the upper part is stayed by the cover-plate. Then, for the purpose of proportioning the latticing, the section may be considered to consist of

2 web-plates	$9\text{x}\ \frac{3}{8}$ ins.	=	6.75 (14 ins. out to out)
2 flange-angles	$6\text{x}3\frac{1}{2}\text{x}\frac{7}{16}$ "	=	7.94 (6-in. legs vertical)
			14.69 sq. ins.

The distance from the neutral axis (through the center of gravity of the section and parallel with the web-plates) to the center of gravity of either rib = 7.34 ins.; the distance to the outer fibers = 10.5 ins.; the radius of gyration about this axis = 7.57 ins.; and the length of the member = 25 ft. Hence $\frac{l}{r} = \frac{25}{7.57} = 3.3$, which (by Table I, Chap. I) corresponds with a permissible unit-stress of 14,720 lbs. per sq. in. for square or fixed ends. Now

$A = 14.69$ sq. ins

$x = 7.34$ ins.

n = 10.5 ins.
p = 16,000 lbs. per sq. in.
p_1 = 14,720 lbs. per sq. in.

$$d = (16{,}000 - 14{,}720) \times \frac{7.34}{10.5} = 895 \text{ lbs. per sq. in.}$$

$$i = \frac{2 \times 14.69 \times 875}{25} = 1{,}050 \text{ lbs. per lin. ft. of column.}$$

The latticing is single and at an angle of 60° with the longitudinal axis of member; thus it will be found that each bar covers a length of 0.86 ft.; and, since the latticing is on one side only, $s = i\,y \sec \alpha = 1{,}050 \times 0.86 \times 2 = 1{,}800$ lbs., which is the stress in each bar. The drawing (Fig. 12) calls for 2½x⅜-in. lattice-bars. The area of one bar = 0.94 sq. in.; its length c. to c. of rivets = 1.73 ft.; and its least radius of gyration = 0.11 in. Then $\frac{l}{r} = \frac{1.73}{0.11} = 15.7$, which corresponds with a permissible unit-stress of 5,400 lbs. per sq. in. for square ends; and, since the area of a bar = 0.94 sq. in., its value in compression = $5{,}400 \times 0.94 = 5{,}070$ lbs., being much greater than the stress as figured above.

The latticing of the main posts of the Railway Viaduct (Chap. VII) will now be investigated. The posts are constructed of two 18-in. I-beams @ 55 lbs. = 31.86 sq. ins., spaced 15.5 ins. c. to c., and their greatest unsupported length is about 30.5 ft. The distance from the neutral axis (through the center of gravity and parallel with the webs of I-beams) to the center of gravity of either rib = 7.75 ins.; the distance to the outer fibers = 10.75 ins.; the radius of gyration about this axis = 7.85 ins.; and $\frac{l}{r} = \frac{30.5}{7.85} = 3.9$. The specification requires that the pin-end formula shall be used in designing trestle posts, which is not on account of their being less firmly fixed at the panel-points than many other compression members assumed to have square ends, but because of the somewhat unsymmetrical application of the load. Therefore, to be consistent, the same formula should be employed in determining the stress to be transmitted by the lattice-bars, assuming that this stress is

transmitted in one-quarter of the unsupported length of the member, the same as for any other square ended column. By reference to Table I, the permissible average unit-stress is found to be 12,870 lbs. per sq. in. Then

A = 31.86 sq. ins.
x = 7.75 ins.
n = 10.75 ins.
p = 16,000 lbs. per sq. in.
p_1 = 12,870 lbs. per sq. in.

$$d = (16{,}000 - 12{,}870) \times \frac{7.75}{10.75} = 2{,}250 \text{ lbs. per sq. in.}$$

$$i = \frac{2 \times 31.86 \times 2{,}250}{30.5} = 4{,}700 \text{ lbs. per lin. ft. of column.}$$

The latticing is single, and inclined at an angle of 60° with the longitudinal axis of the member; but it intersects some distance beyond the center of gravity of the individual ribs, thus permitting of secondary bending moments therein. These bending moments would be greatly reduced if the ends of the lattice-bars overlapped instead of abutting, as shown in Fig. 21. It is seldom practicable to design the latticing so that it will intersect exactly at the center of gravity of the ribs connected thereby, but this condition should be realized as near as possible; and, further, the lattice-bars should be connected directly to the tie-plates without any intervening space. By referring to Fig. 21, it will be found that each lattice-bar covers 1.5 ft. of the post; and, since the latticing is on two sides, the longitudinal component of the stress in one bar $= \frac{4{,}700 \times 1.5}{2} = 3{,}525$ lbs. The stress in each bar is equal to this latter figure multiplied by the secant of 60°, $= 3{,}525 \times 2 = 7{,}050$ lbs. The latticing shown is $5 \text{x} \frac{7}{16}$ ins. The area of one bar = 2.19 sq. ins., its radius of gyration = 0.126 in.; and its length c. to c. of rivets = 1.9 ft. Then $\frac{l}{r} = \frac{1.9}{0.126} = 15$, which corresponds to the unit stress of 5,710 lbs. per sq. in. for fixed ends; and the value of each bar in compression $= 5{,}710 \times 2.19 = 12{,}500$ lbs., being greatly in excess of the stress attributed to it.

The next example will be a heavy column of comparatively small general dimensions, made up as follows:

2 web-plates	24x$\frac{7}{8}$ in.	=	42.00 (16 ins. out to out)
4 flange-angles	6x4x$\frac{7}{8}$ "	=	31.96 (6-in. legs vertical)
2 filler-plates	12x$\frac{7}{8}$ "	=	21.00 (between flange-angles)
2 outside plates	22x$\frac{7}{8}$ "	=	38.50
			133.46 sq. ins.

The distance from the neutral axis (through the center of gravity of the section and parallel with the web-plates) to the center of gravity of either rib = 8.55 ins.; the distance to the outer fibers = 12 ins.; the radius of gyration about this axis = 8.33 ins.; and the length of the column, which is assumed to be fixed at the ends, = 30 ft. Now $\frac{l}{r} = \frac{30}{8.33} = 3.6$, which corresponds to the unit-stress of 14,500 lbs. per sq. in. Then

A = 133.46 sq. ins.
x = 8.55 ins.
n = 12 ins.
p = 16,000 lbs. per sq. in.
p_1 = 14,500 lbs. per sq. ins.

$$d = (16,000 - 14,500) \times \frac{8.55}{12} = 1,070 \text{ lbs. per sq. in.}$$

$$i = \frac{2Ad}{l} = \frac{2 \times 133.46 \times 1,070}{30} = 9,550 \text{ lbs. per lin. ft. of column.}$$

With latticing at 60°, the length of column covered by each bar = 1 ft. = y; and, since the column is latticed on two sides,

$$s = \frac{iy \sec \alpha}{2} = \frac{9,550 \times 1 \times 2}{2} = 9,550 \text{ lbs.},$$

which is the stress in each bar The length of the lattice-bars c. to c. of rivets = 2 ft. = 24 ins., and their required thickness (by Art. 23 of specification) = 24/50 = $\frac{1}{2}$ in. Their width is not specified, but it will be assumed to be 5 ins. Now, the area of one bar = 2.5 sq. ins.; its least radius of gyration = 0.145 in.; and $\frac{l}{r} = \frac{2}{0.145} = 13.8$, which corresponds to the unit-stress of

6,340 lbs. per sq. in. for fixed ends. Then $6{,}340 \times 2.5 = 15{,}850$ lbs., which is the value of one bar in compression. The ends of these bars should lap over one another, and they should be connected to the flange-angles by two $\frac{7}{8}$-in. rivets passing through two adjacent bars.

In all of the foregoing examples it has been shown that the latticing of compression membeis of moderate dimensions, proportioned in accordance with the specification contained in Chapter I, is of ample strength; and it is reasonable to suppose that equally satisfactory results may be obtained for members of any size by employing the method of proportioning the lattice-bars herein set forth.